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April 30, 2018

Moon Wu
5660 East Mercer Way
Mercer Island, WA 98040

Geotechnical Engineering Evaluation
East Mercer Way Retaining Walls and Slope Stabilization
5660 East Mercer Way
Mercer Island, Washington
NGA File No. 1024718

Dear Moon Wu,

This report summarizes the results of our geotechnical engineering evaluation and stabilization recommendations of the steep slopes and existing block retaining walls located at your residence located at 5660 East Mercer Way on Mercer Island, Washington, as shown on the Vicinity Map in Figure 1. Our services were completed in general accordance with our services agreement signed by you on January 17, 2018.

INTRODUCTION

The purpose of this study is to explore and characterize the surface and subsurface conditions within the vicinity of the existing block retaining walls and steep slopes in order to provide our opinions and recommendations with respect to the stabilization of the slope and retaining wall system.

We initially visited the site on December 11, 2017 to observe the existing site conditions. We understand and observed that a series of tiered concrete block retaining walls were constructed within a steep east-facing slope area below and to the east of the existing residence. We were informed that these walls were constructed without a permit and the City of Mercer Island has requested a geotechnical evaluation be performed prior to approving the existing wall construction or proposed stabilization measures. You have requested that we explore the site within the vicinity of the lower steep east-facing slope and the block retaining walls and provide our opinion regarding the stability of the existing block walls, and to provide recommendations for potential repairs or improvements to the walls.

SCOPE

The purpose of this study is to explore and characterize the site subsurface conditions and provide recommendations for stabilizing affected areas. Specifically, our scope of services included the following:

1. A review of available soil and geologic maps of the area.
2. Exploring the subsurface soil and groundwater conditions within the eastern portion of the residence and in the vicinity of the retaining walls using a limited-access drill rig and hand auger explorations. Drill rig was subcontracted by NGA.
3. Mapping the conditions on the sloping areas below the residence and evaluate current slope stability conditions.
4. Providing recommendations for permanently stabilizing the affected areas.
5. Providing our opinion regarding the construction and stability of the existing block retaining walls.
6. Providing recommendations for potential retaining wall repairs or improvements.
7. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site consists of a roughly rectangular-shaped parcel covering approximately 0.42 acres. The site is occupied by a multi-story, single-family residence adjacent to East Mercer Way. Moderate to steep slopes exist throughout the property, occupying areas to the east of the residence that descend from the eastern side of the residence to adjacent properties along a lower private road. The tiered block walls were constructed along the surface of the steep slope below and to the east of the residence. The property is bordered to the north, south, and east by existing single-family residences, and to the west by East Mercer Way. The site layout within the vicinity of the residence is shown on the Schematic Site Plan in Figure 2.

A series of tiered block retaining walls is located below the residence along a steep east-facing slope that descends to an adjacent residential property below. The steep east-facing slopes steps down at gradients in the range of 20 to 36 degrees (36 to 73 percent grade). A profile of the existing ground surface through the block wall area, and the interpreted subsurface conditions within the steep slope are presented in Cross Sections A-A’ and B-B’ in Figure 3 and 4, respectively. We observed that portions of the block retaining walls have experienced minor distress since construction, as they appear to be bowing in some areas. The walls range from approximately 3.1 to 4.1 feet in exposed height and are separated as little as five feet of horizontal distance. The base of the walls appear to not be embedded and no geogrid reinforcement was utilized in the wall construction. The overall height of the slope and tiered retaining walls below the residence is approximately 22 feet. The slope outside the retaining wall area is generally vegetated with underbrush and sparse trees. We did not observe any indications of past sloughing events on the steep slopes outside of the retaining wall area. We also did not observe surface or seeping water in the immediate vicinity of the residence or on the slope during our site visit on February 20, 2018.

Subsurface Conditions

Geology: The geologic units for this area are shown on the Geologic Map of Mercer Island, Washington, by Kathy G. Troost and Aaron P. Wisher (GeoMapNW and the City of Mercer Island, 2006). The project site is mapped as surficial deposits of the Fraser Glaciation, consisting of Vashon Stade glacial advance outwash (Qva) and the Lawton Clay (Qvlc). The advance outwash deposits are described as well-sorted sand and gravel deposited by streams flowing from the advancing ice sheet. The clay is described as massive silt and clay with scattered drop-stones, which was deposited in lakes dammed by continental glaciation in the last Ice-Age. Our explorations generally encountered undocumented fill underlain by fine to medium grained sand with varying amounts of silt with gray, fine sandy silt in the lower portion of the site, consistent with the description of advance outwash and the Lawton Clay mapped in this area, respectively.

Explorations: The subsurface conditions within the site were explored on February 20, 2018 by drilling three borings with a limited-access drill rig extending 16.5 feet below the existing ground surface within the east-facing steep slope. The approximate locations of our explorations are shown on the Schematic Site Plan in Figure 2. A geologist from Nelson Geotechnical Associates, Inc. (NGA) was present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the explorations.

For the borings, a Standard Penetration Test (SPT) was performed on each of the samples during drilling to document soil density at depth. The SPT consists of driving a 2-inch outer-diameter, split-spoon sampler 18 inches using a 140-pound hammer with a drop of 30 inches. The number of blows required to drive the sampler the final 12 inches is referred to as the "**N**" value and is presented on the boring logs. The **N** value is used to evaluate the strength and density of the deposit.

The soils were visually classified in general accordance with the Unified Soil Classification System presented in Figure 5. The logs of our borings are attached to this report and are presented as Figures 6 through 8. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the boring logs should be reviewed.

In all of our borings, we encountered approximately 1.0 to 4.0 feet of surficial undocumented fill containing fragments of brick. Below the surficial fill, all explorations encountered between 9.0 and 15.0 feet of loose to medium dense, tan to gray-brown, silty, fine to medium sand with varying amounts of iron oxidation staining and gravel, which was interpreted to be the native advance outwash deposits. Underlying the advance outwash in Borings 2 and 3, we encountered hard, blue-gray to gray, fine sandy silt that we interpreted as native Lawton Clay soils at depths of 13.0 below the surface. Borings 1, 2, and 3 were terminated within the native glacial soils at depths of 16.5 feet below the existing ground surface.

Hydrologic Conditions

We did not encounter groundwater seepage in any of the explorations completed during fieldwork. If groundwater were to be encountered within this site, we would consider this condition to be perched groundwater. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of underlying, less permeable soils. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of precipitation. We would expect the amount of perched water to decrease during drier times of the year and increase during wetter periods.

SENSITIVE AREA EVALUATION

Seismic Hazard

The 2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Since medium stiff to hard soils were generally encountered underlying the site at depth, the site conditions best fit the IBC description for Site Class C. Table 1 below provides seismic design parameters for the site that are in conformance with the 2015 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – 2015 IBC Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	1.445	0.554	1.00	1.512	0.963	0.554

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. It is our opinion that the competent native soils interpreted to underlie the site have a low potential for liquefaction or amplification of ground motion.

The medium dense or better soils interpreted to form the core of the site slope are considered stable with respect to deep-seated slope failures. However, the loose surficial materials and undocumented fill on the slope, if not removed or suitably stabilized, have the potential for failures during seismic events. Such events should not directly affect the existing residence provided the recommended repairs to the residence and slope stabilization measures are designed and implemented as described in this report.

Landslide Hazard/Slope Stability

The criteria used for evaluation of landslide hazards includes soil type, slope gradient, and groundwater conditions. Steep east-facing slopes with a gradient of up to approximately 20 to 36 degrees (36 to 73 percent) with a height of approximately 22 feet, is located immediately below the existing residence. We observed minor signs of distress within the retaining walls such as bowing. We did not observe significant indications of distress within the residence foundation.

Our explorations and observations indicate that the core of the steep slope below the fill consists primarily of competent glacial outwash soils. It is our opinion that the core of the slope is stable and that the recommended block wall repairs should terminate in stable soils. It is also our opinion that there is a significant potential for on-going failures within the loose surficial and undocumented fill soils on the steep slope if these soils are not stabilized. Proper site grading and drainage as well as stabilization techniques as recommended in this report should help improve current stability conditions. We also recommend that the slope be continually monitored for any indications of instability and stabilization measures be implemented immediately if they are observed.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion from a geotechnical standpoint that the existing block walls on the steep slope below the residence were not adequately installed and/or engineered, and are failing due to a combination of several factors. These factors include: lack of adequate drainage measures behind the walls, lack of geogrid reinforcement, supporting the walls on unsuitable material, inadequate wall toe embedment, and placement of unsuitable fill behind the walls. We observed drainage system components including drain pipes and drain rock layers behind some of the retaining walls, probably placed after construction of the walls. Our explorations encountered between approximately 1.0 and 4.0 feet of loose undocumented fill soils that are not suitable as structural fill immediately below and surrounding some of the walls. Multitiered retaining wall systems need to have an engineered design and need to utilize geogrid reinforcement to support the backfill material. We understand that an engineered design was not used in the construction of the walls.

To restore the stability of the steep slope area below the residence, we recommend removing all of the concrete block retaining walls from the steep slope area and reconstructing a tiered retaining wall system with the provided design. The new geogrid-reinforced fill walls could be constructed using the existing retaining wall blocks or using new Keystone Compaq blocks. Loose native and undocumented fill soils are interpreted to underlie the slope areas that are not suitable for support of the proposed retaining walls. We recommend that the base of the new wall blocks and reinforced fill area be supported directly on competent native soils. The base of the new walls should be embedded a minimum of 18-inches below the finished ground surface.

We anticipate that the individual tier will have a maximum exposed height of approximately 8.0 feet but may be higher depending on actual site elevations. We anticipate that the total wall height may be up to ten feet in order to satisfy a recommended base embedment of 18 inches below finished ground surface. This is discussed further in the **Wall Design and Construction Recommendations** subsection of this report. Due to the tight site constraints and the substantial amount of fill material that will need to be removed from the wall and reinforced fill area prior to construction of the walls, we stress that implementing proper planning and construction staging techniques will be key to achieve a successful outcome. NGA should be retained to review project plans prior to construction and should be retained to observe wall construction to verify wall installation is being performed in accordance with the plans and our recommendations provided in this report.

All residence drains including roof, driveway, footing, and yard drains along with drains associated with the proposed wall construction should be thoroughly investigated and directed to flow into an approved system. All existing drain pipes within the steep slope area should be abandoned and removed as a part of the drainage improvements.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface water or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the soil and groundwater conditions encountered and able to monitor the nature and condition of the cut slopes.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V). If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. The slope protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

Permanent cut and fill slopes should be no steeper than 2H:1V. However, flatter inclinations may be required in areas where loose soils are encountered. If permanent slopes steeper than 2H:1V are created, we would anticipate such slope(s) to require on-going maintenance. Permanent slopes should be planted and the vegetative cover should be maintained until it is established. We should review plans and visit the site to evaluate excavations for this project.

Slope Improvements

Geogrid-Reinforced Block Wall Design and Construction: The total height of the planned replacement walls is expected to be up to approximately 10 feet, including a minimum recommended embedment of 1.5 feet below the finished ground surface. We have provided wall designs for a tiered wall system with an individual tier height up to a 10-foot high retaining wall with Keystone block facing or utilizing the existing blocks on site. We recommend that walls be constructed utilizing geogrid reinforced backfill. The wall detail and design parameters along with construction notes are shown on Figure 9. Keystone Block wall calculations are provided in Appendix A. We have assumed that the retained fill zones will consist of granular material compacted to structural fill specifications. We understand that the fill will be placed level behind the walls and extending back into the slope. As indicated on the detail, the drainage system should be installed along the base of the blocks.

The block facing should consist of Keystone Compaq blocks or the existing blocks on site. The block facing should be placed on a minimum of 4-inch thick crushed rock leveling pads placed over competent native soils, or structural fill material prepared under the supervision of NGA. Unsuitable undocumented fill soils will likely be encountered at the retaining wall subgrades. We recommend that the wall and reinforced-fill subgrade be extended down to expose competent native soils. The wall and reinforced fill areas should also be graded to level benches prior to wall and reinforced fill construction. Since the walls will be terraced, we recommend that the lowest block retaining wall be constructed to completion prior to beginning construction of the upper walls. All tiers should be separated by a minimum horizontal distance that equals the total height of the tier below.

A drainage blanket of 12 inches of free-draining crushed rock should be placed between the blocks and the retained fill zone. The block cavities should also be filled with the crushed rock. A rigid, perforated drainpipe embedded in a minimum of 1-foot of pea gravel and wrapped in a filter fabric should be placed at the bottom of the drainage blanket. The drain should be sloped to drain into a permanent discharge point placed at the bottom of the slope.

Stratagrid SG500 geogrid (or equivalent) is recommended in the wall designs. The geogrid should be cut to the recommended lengths, attached to the blocks as recommended by the manufacturer, and extended back into the reinforced fill zone. The grid should be pulled tight before the fill is placed over the geogrid. Care should be taken not to damage the geogrid by operating construction equipment on the exposed grid, or by allowing large rocks to be placed directly on the grid.

All fill placed in the retained fill zone behind the retaining walls should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The fill subgrade should consist of native medium dense or better native soil compacted to a non-yielding condition. The fill subgrade should consist of level benches.

Structural fill should consist of a good quality, granular soil, free of organics and other deleterious material and be well graded to a maximum size of about three inches. The material should have no more than 10 percent by weight of the portion passing the US #200 Sieve. We should be retained to evaluate proposed fill material prior to construction.

Following subgrade preparation, placement of structural fill may proceed. All fill placements should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill should be compacted to a minimum of 95 percent of the material’s maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D 1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Site Drainage

If ground water seepage is encountered or if excessive rainfall occurs during construction of specific aspects, we recommend that the contractor slope the bottom of the excavations and direct the water to ditches and small sump pits. The collected water can then be directed to a suitable discharge point at the bottom of the slope. The recommended retaining walls should have drains as described in the previous section.

We also recommend that all residence downspouts and yard drains be investigated to understand where they are directed. All drain pipes within the steep slope area should be abandoned and removed. If any irrigation systems are located within the steep slopes they should also be abandoned and removed. We recommend that all of the existing roof, footing, yard, and driveway drains associated with the residence be tightlined to flow into an approved system. NGA should be retained to evaluate the drainage systems as they are investigated and constructed.

CLOSURE

Based on our understanding of the proposed plans, and provided that the recommendations in this report are strictly followed during construction, the areas disturbed by construction should remain stable. The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe meeting the requirements stated in Mercer Island City Code 19.07.060.D.2.a. Therefore, the risk of damage to the proposed development or to adjacent properties from soil instability should be minimal, and the proposed grading and development should not increase the potential for soil movement.

USE OF THIS REPORT

NGA has prepared this report for Ms. Moon Wu and her agents, for use in the planning and design of the slope and residence stabilization project on this site only. This letter is a specific evaluation of the observed soil settlement and related distress, and the existing concrete block retaining walls. The scope of our work does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explored and unexplored areas and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

All people who own or occupy homes on hillsides should realize that landslide movements are always a possibility. The landowner should periodically inspect the slope, especially after a winter storm. If distress is evident, a geotechnical engineer should be contacted for advice on remedial/preventative measures. The probability that landsliding will occur is substantially reduced by the proper maintenance of drainage control measures at the site (the runoff from the roofs should be led to an approved discharge point). Therefore, the homeowner should take responsibility for performing such maintenance. Consequently, we recommend that a copy of our report be provided to any future homeowners of the property if the home is sold.

We recommend that NGA be retained to review final plans prior to construction. We also recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

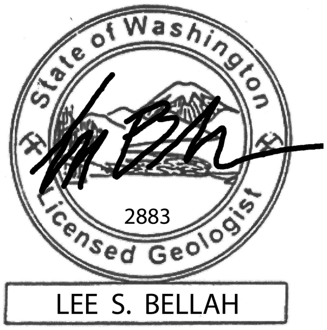
o-o-o

It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Carston T. Curd, GIT
Staff Geologist



Lee S. Bellah, LG
Project Geologist

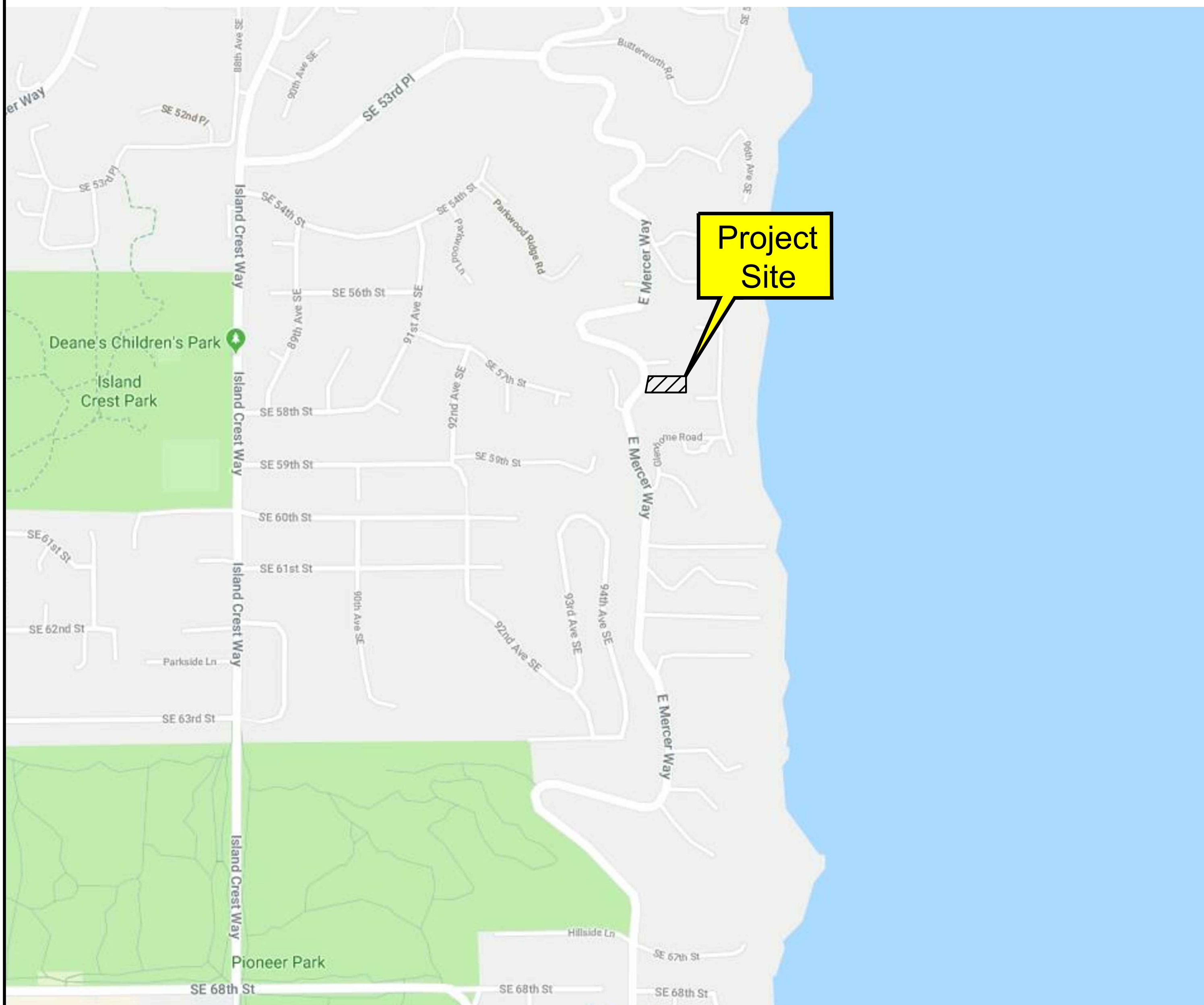
Khaled M. Shawish, PE
Principal

CTC:LSB:KMS:dy

Nine Figures and Appendix A Attached

VICINITY MAP

Not to Scale



Mercer Island, WA

Project Number	Wu Retaining Wall Vicinity Map	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
1024718			1	3/8/18	Original	DPN	LSB
Figure 1							

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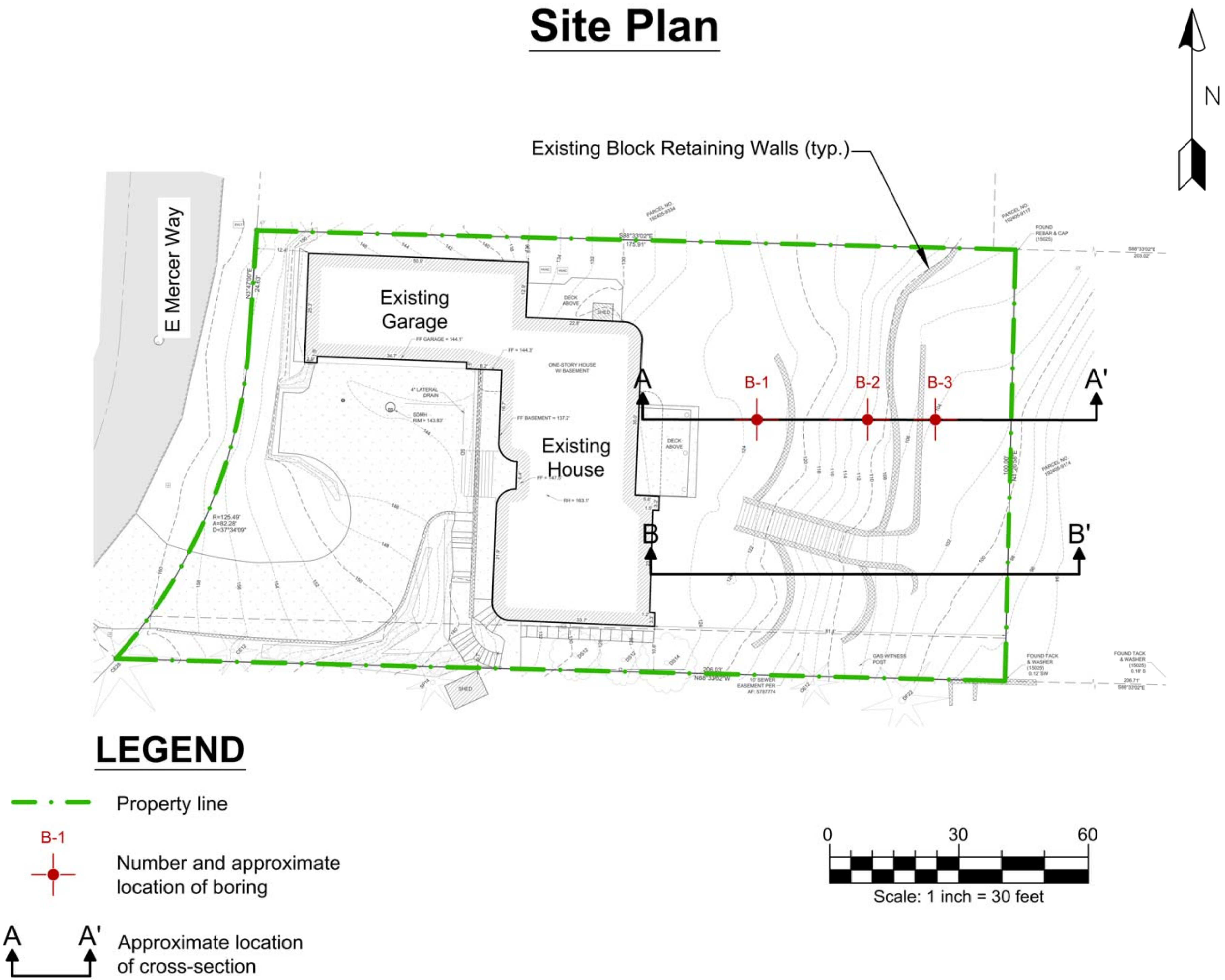
Site Plan



Reference: Site plan based on a plan dated August 29, 2017 titled "Jake Zhen Lin Li and Moon Yanci Wu," prepared by Site Surveying, Inc.

Project Number 1024718

Figure 2

Wu Retaining Wall Site Plan

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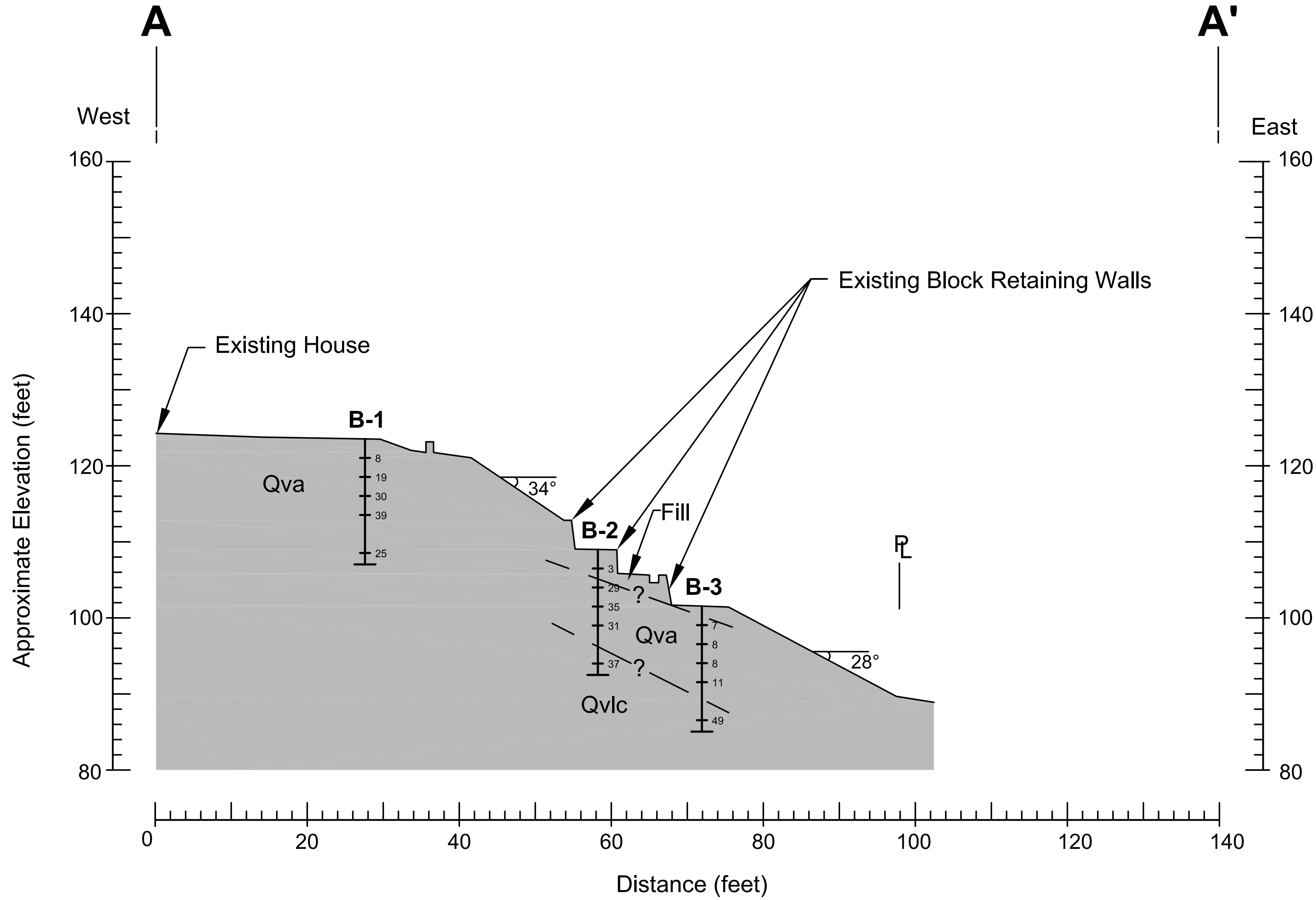
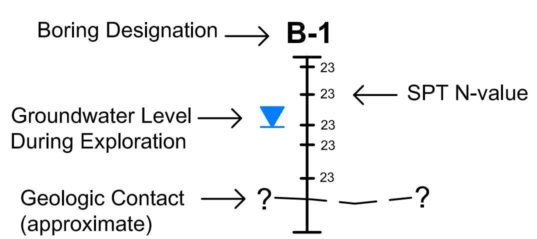


Exploration



NOTES:
1) Stratigraphic conditions are interpolated between the explorations. Actual conditions may vary.
2) Elevations are approximate.

Reference: Cross Section is based on field measurements using a hand-held clinometer and 100-ft tape measure.

Figure 3

Project Number 1024718

Wu Retaining Wall
Cross-Section A-A'

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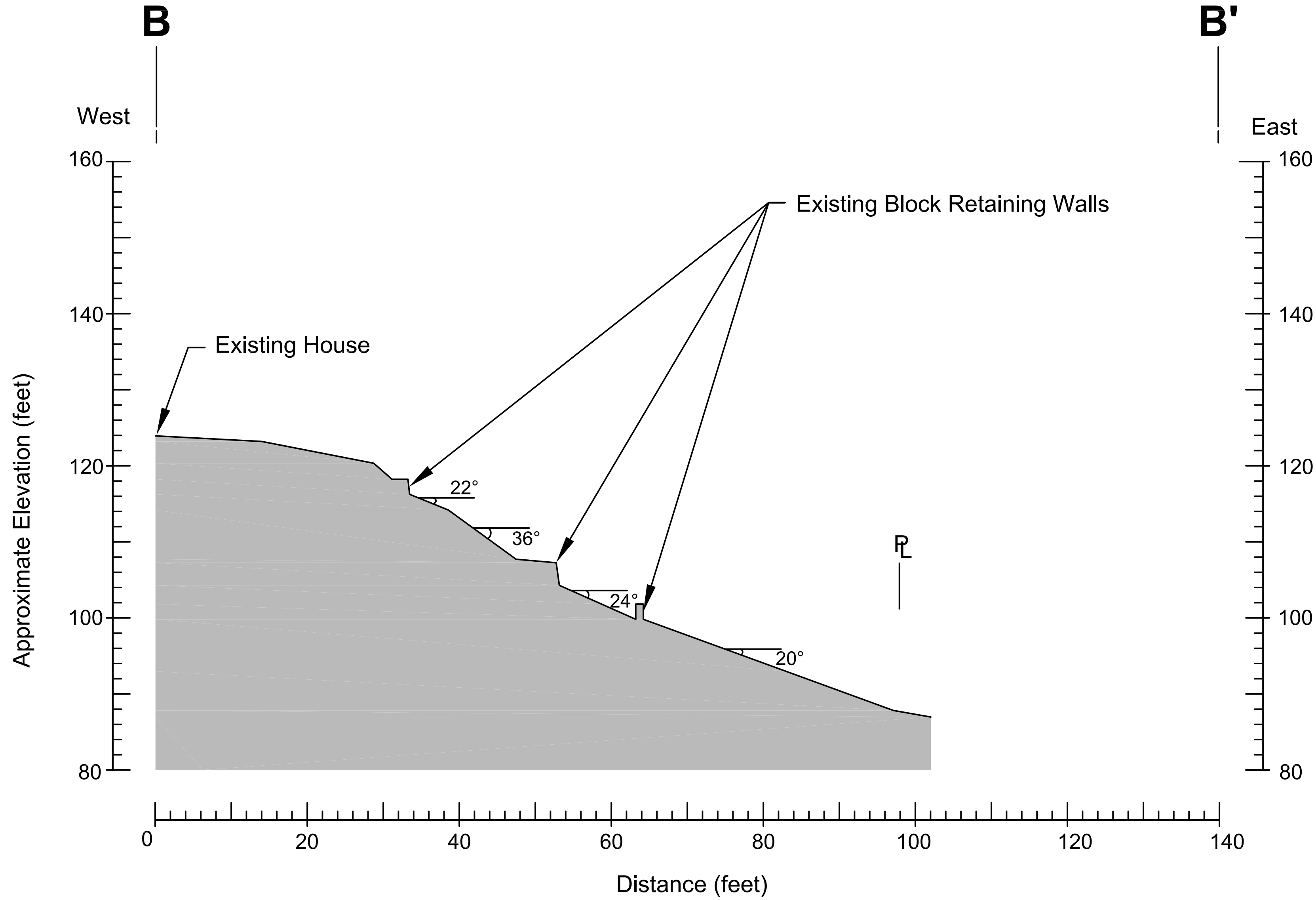
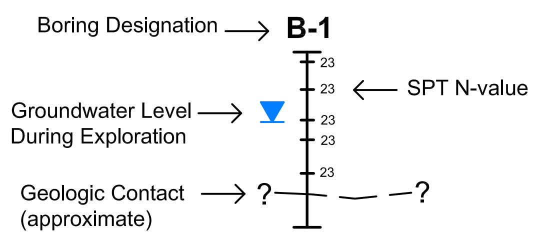
Project Number
1024718

Figure 4

Wu Retaining Wall
Cross-Section B-B'



Exploration



NOTES:
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2) Elevations are approximate.

Reference: Cross Section is based on field measurements using a hand-held clinometer and 100-ft tape measure.

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number 1024718	Wu Retaining Wall Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 5			1	3/8/18	Original	DPN	LSB

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BORING LOG

B-1

Approximate Ground Surface Elevation: 124 ft

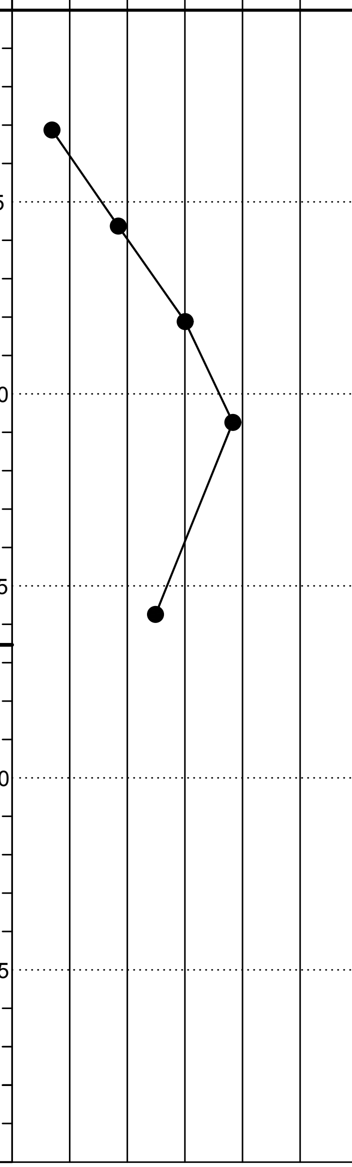
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (Depth in feet)
Dark brown, silty fine to coarse sand with gravel, brick, and charcoal (loose, moist) (**FILL**)				
Tan, fine to medium sand with silt and iron-oxide staining (loose, moist)		SP-SM	8	
-becomes gray, medium dense			19	5
-becomes medium dense to dense			30	
-becomes dense, dry			39	10
-becomes medium dense			25	15
Boring terminated below existing grade at 16.5 feet on 2/20/18. Groundwater seepage was not encountered during drilling.				20, 25

Penetration Resistance (Blows/foot - ●) 10 20 30 40 50 50+

Moisture Content (Percent - ■) 10 20 30 40 50 50+

Laboratory Testing

Piezometer Installation - Ground Water Data (Depth in Feet) 5 10 15 20 25



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number 1024718	Wu Retaining Wall Boring Log
Figure 6	
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No.	Date	Revision	By	CK
1	3/9/18	Original	DPN	LSB

Logged by: CTC on 2/20/2018

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BORING LOG
B-2

Approximate Ground Surface Elevation: 114 ft

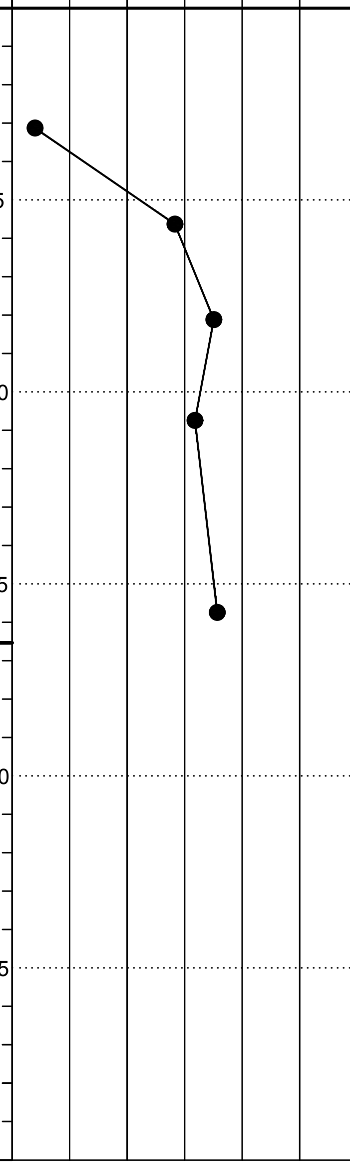
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (Depth in feet)
Dark brown, silty fine to coarse sand with gravel, brick, and charcoal (very loose, moist) (**FILL**)				
-becomes wet			3	
Gray-brown, silty fine to medium sand (medium dense, moist)		SM	29	5
-becomes tan, silty fine sand with iron-oxide staining and trace gravel, dense		SM	35	
-becomes brown to gray		SM	31	10
Gray, fine sandy silt with trace coarse sand (hard, dry)		ML	37	15
Boring terminated below existing grade at 16.5 feet on 2/20/18. Groundwater seepage was not encountered during drilling.				20, 25

Penetration Resistance (Blows/foot - ●) 10 20 30 40 50 50+

Moisture Content (Percent - ■) 10 20 30 40 50 50+

Laboratory Testing

Piezometer Installation - Ground Water Data (Depth in Feet): 5, 10, 15, 20, 25



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

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Project Number	Wu Retaining Wall Boring Log	NELSON GEOTECHNICAL ASSOCIATES, INC.
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BORING LOG

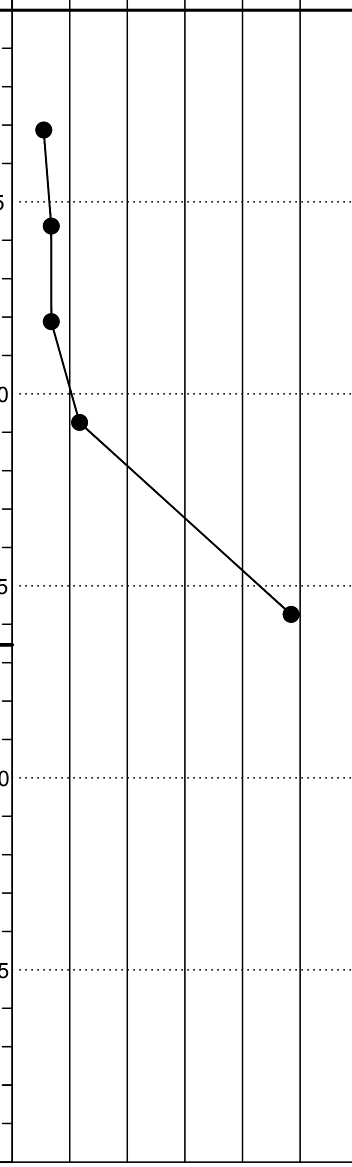
B-3

Approximate Ground Surface Elevation: 104 ft

Description	Graphic Log	Group Symbol	Blow Count	Sample Location (Depth in feet)
Dark brown, silty fine to coarse sand with gravel, brick, and charcoal (very loose, moist) (**FILL**)				
Tan, fine to medium sand with silt and iron-oxide staining (loose, moist)		SP-SM	7	
Gray-brown, silty fine to medium sand with trace iron-oxide staining (loose, moist)			8	
-with trace organic debris		SM	8	
-becomes gray, medium dense			11	
Blue-gray silt with clay and trace fine sand (hard, dry)		ML	49	
Boring terminated below existing grade at 16.5 feet on 2/20/18. Groundwater seepage was not encountered during drilling.				

Penetration Resistance (Blows/foot - ●): 10, 20, 30, 40, 50, 50+

Moisture Content (Percent - ■): 10, 20, 30, 40, 50, 50+



Laboratory Testing

Piezometer Installation - Ground Water Data (Depth in Feet): 5, 10, 15, 20, 25

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

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Project Number 1024718	Wu Retaining Wall Boring Log
Figure 8	
Page 1 of 1	

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1	3/9/18	Original	DPN	LSB

Logged by: CTC on 2/20/2018

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SPECIFICATIONS FOR REINFORCED WALL

General
1. The contractor shall have an approved set of plans and specifications on site at all times during the construction of the wall. The wall layout is the responsibility of the contractor.
2. Nelson Geotechnical Associates (NGA) shall observe and monitor the construction of the wall on a full-time basis.
3. Stratagrid SG500 geogrid or equivalent shall be used for this project. All geogrid and facing materials shall be approved by NGA prior to installation.
4. The contractor may use longer geogrid lengths than the design sections for ease of construction. The geogrid lengths may not be shorter unless approved by NGA.

Subgrade Preparation
1. The ground shall be prepared by removing surficial organics, loose soil and undocumented fill to expose competent native soils as approved by NGA.
2. Exisitng utilities shall be located and their depths varified in the field. If utility trenches or undocumented fill are encountered within the wall or reinforced fill subgrade, the subgrade shall be prepared as recommended and approved by NGA.
3. A generally level bench with a minimum width equal to the design length of the geogrid is required for placement fo the reinforced fill.
4. The excavation shall be cleaned of all excess material and protected, as necessary, from construction traffic to maintain the intergrity of the subgrade.
5. The wall and reinforced fill subgrade should expose competent native soils. Subgrades to be approved by NGA.
6. The base of the excavation should be deep enough to satisfy a minimum embedment of 1.5 feet. The wall shall also be deep enough to satisfy a minimum distance of a 1H:1V inclination between the base of the upper block wall and the base of the lower block wall.
7. The excavation walls shall be sloped back at 1.5H:1V for safety. If this is not feasible, specific recommendations for maintaining excavation stability shall be provided by NGA. All WISHA/OSHA safety requirements shall be observed at all times during construction.

Geogrid Placement
1. The reinforcement shall be rolled out, cut to length, and laid at the proper elevation, location, and orientation. Orientation of the reinforcement is of extreme importance since geogrids vary in strength with roll direction. The contractor shall be responsible for the correct orientation.
2. Geogrid shall be placed at the location and elevations shown on the plans. The geogrid length is measured from the face of the blocks.
3. Prior to placing the fill, the geogrid shall be pulled to remove the slack and stretched by hand until taut and free of wrinkles.

Fill Placement
1. Structural fill, consisting of granular import soils or granular on-site material no greater than 3 inches in size, shall then be placed upon the subgrade and geogrid. If larger rock is used in the fill, additional layers of geogrid may need to be used in the reinforcement. The contractor shall prevent damage to the geogrid by placing the first lift of structural fill with at least a 1-foot thickness. NGA shall approve material placed in the reinforced zone, before placement.
2. Structural fill shall have parameters equal to or better than those stated for the reinforced wall fill below with less then 20 percent passing the number 200 sieve. NGA may allow a higher silt content based on review of the wall design and proposed fill parameters.
3. Soil density tests shall be performed as designated by NGA.
4. Fill soils in the wall area shall be compacted to at least 95 percent of the Maximum Dry Density (MDD) as determined by ASTM D-1557.
5. The soil shall be placed in relatively uniform horizontal lifts not exceeding 10 or 12 inches in thickness. The lift thickness shall not exceed the manufacturer's recommended depth for the compactive device used on the project.

Drainage
1. A specific drainage system is shown on the plans. Alternative drains can be used based on conditions found in the field and the material used within the reinforced zone. Changes to the drainage system should be approved by NGA prior to placement.
2. A drainage blanket 12 inches in width shall be installed directly behind the block facing and shall consist of 2-inch clean crushed rock. All of the drainage materials shall have a fines content no greater than 5 percent passing the number 200 sieve. A 4-inch rigid perforated pipe embedded in a minimum of one foot of pea gravel or washed rock and wrapped with filter fabric shall be installed at the bottom of the drainage layer
3. Surface water shall not be allowed to pond in or near the reinforced fill zone during or after construction.
4. Suitable clean-outs shall be installed every 50 feet for future maintenance.
5. Surface water shall not be allowed to reach the drainage layer.

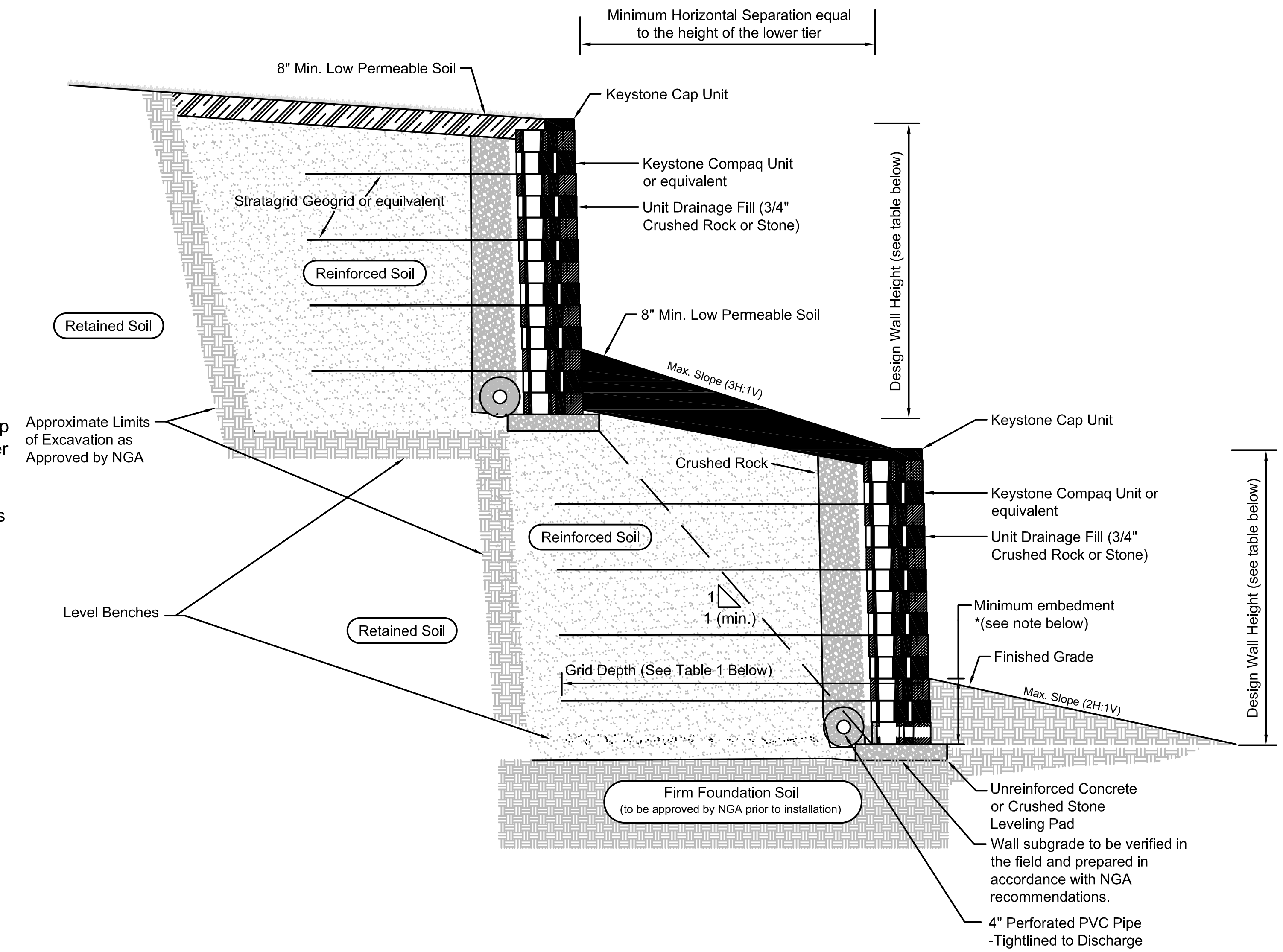
Design Parameters
Reinforced Wall Fill: 30 degrees, 0 PSF, 120 PCF
Retained Backfill: 30 degrees, 0 PSF, 120 PCF
Foundation Soil: 30 degrees, 0 PSF, 120 PCF

Seismic
0.2g peak ground acceleration

External Stability of Wall
Minimum Factor of Safety against Base Sliding: 1.5
Minimum Factor of Safety against Overturning: 2.0
Minimum Factor of Safety against Bearing Capacity: 2.0

Internal Stability of Wall
Minimum Factor of Safety on Geogrid Strength: 1.5
Minimum Factor of Safety on Geogrid Pullout: 1.5
Soil-Geogrid Interaction Coefficient: 1.0
Percent Coverage of Geogrid: 100 Percent

Inspection
Wall construction shall be completed under the direction of NGA.



*Note: Minimum wall embedment shall be 1.5 feet or greater to maintain the 1H:1V seperation between the bottom of the upper block wall and back of the lower block walls, as shown above.

Table 1:

Wall Height (feet)	Number of Geogrid Layers	Geogrid Length (feet)	Geogrid Height Above Leveling Pad/ Geogrid Type (feet)				
4	2	5.0	0.67 SG 500*	2.67 SG 500			
6	3	6.0	0.67 SG 500	2.67 SG 500	4.67 SG 500		
8	4	7.0	0.67 SG 500	2.67 SG 500	4.67 SG 500	6.67 SG 500	
10	5	9.0	0.67 SG 500	2.67 SG 500	4.67 SG 500	6.67 SG 500	8.67 SG 500

*Stratagrid SG 500 (or equivalent)

No.	Date	Revision	By	CK
1		Original	LSB	KMS

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East Mercer Way Retaining Wall Slope Stabilization
Geogrid Reinforced Block Wall Detail

Project Number
1024718

Figure 9

APPENDIX A

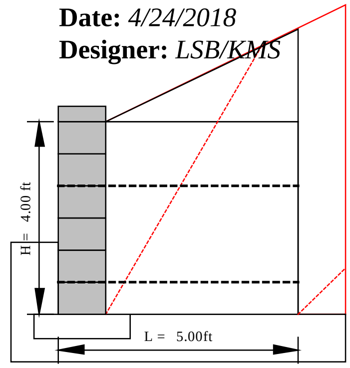
Keystone Block Retaining Wall Calculations

RETAINING WALL DESIGN

KeyWall_2012 Version 3.7.2 Build 10

Project: *5660 East Mercer Way Walls*
Project No: *1024718*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Date: *4/24/2018*
Designer: *LSB/KMS*



Design Parameters

Soil Parameters:	**ϕ deg**	**c psf**	**γ pcf**
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Reinforced Fill Type: *Sand, Silt or Clay*
Unit Fill: *Crushed Stone, 1 inch minus*

Seismic Design A=*0.20 g*, **Kh(Ext)=***0.125* , **Kh(Int)=***0.250* , **Kv=***0.000*

Minimum Design Factors of Safety (seismic are 75% of static)

sliding:	1.50/1.13	pullout:	1.50/1.13	uncertainties:	1.50/1.13
overturning:	2.00/1.50	shear:	1.50/1.13	connection:	1.50/1.13
bearing:	2.00/1.50	bending:	1.50/1.13		

Design Preferences

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	*RFcr*	*RFd*	*RFid*	*LTDS*	*FS*	*Tal*	*Ci*	*Cds*
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Analysis: ***Case: Case 1***
6.0 - foot wall

Unit Type:	*Compac / 120.00 pcf*	*Wall Batter: 0.00 deg (Hinge Ht N/A)*
Leveling Pad:	*Crushed Stone*	
Wall Ht:	*4.00 ft*	*embedment: 1.50 ft*
BackSlope:	*26.00 deg. slope,*	*15.00 ft long*
Surcharge:	*LL: 50 psf uniform surcharge*	*DL: 0 psf uniform surcharge*
	Load Width: 100.00 ft	*Load Width: 100.00 ft*

Results:	***Sliding***	***Overturning***	***Bearing***	***Shear***	***Bending***
Factors of Safety:	*1.95/1.42*	*5.13/3.40*	*14.31/10.87*	*6.12/3.73*	*3.30/0.99<<*

Calculated Bearing Pressure: *688 / 688 / 827 psf*
Eccentricity at base: 0.07 ft/0.39 ft
Reinforcing: (ft & lbs/ft)

Layer	**Height**	**Length**	**Calc. Tension**	**Reinf. Type**	**Allow Ten Tal**	**Pk Conn Tcl**	**Pullout FS**
2	*2.67*	*5.0*	*152 / 279*	*5XT*	*1717/3617 ok*	*802/1070 ok*	*4.80/2.10 ok*
1	*0.67*	*5.0*	*296 / 482*	*5XT*	*1717/3617 ok*	*884/1178 ok*	*5.97/2.93 ok*

Reinforcing Quantities (no waste included):
5XT 1.11 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

DETAILED CALCULATIONS

Project: *5660 East Mercer Way Walls* **Date:** *4/24/2018*
Project No: *1024718* **Designer:** *LSB/KMS*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Soil Parameters:	**ϕ deg**	**c psf**	**γ pcf**
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Leveling Pad: ***Crushed Stone***

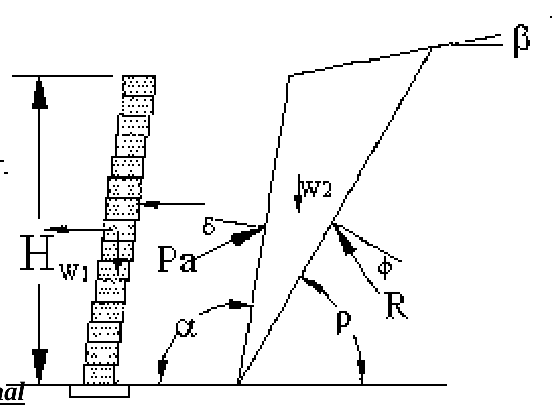
Modular Concrete Unit: *Compac*
Depth: *1.00 ft* **In-Place Wt:** *120 pcf*

Geometry

Internal Stability	*External Stability*
(Sloping geometry)	*(Broken geometry)*
Height: 4.00 ft	*Height: 5.95 ft*
BackSlope:	
Angle: 26.0 deg	*Angle: 26.0 deg*
Height: 7.32 ft	*Height: 5.37 ft*
Batter: 0.00deg	*Batter: 0.00deg*
Surcharge:	
Dead Load: 0.00 psf	*Dead Load: 0.00 psf*
Live Load: 0 psf	*Live Load:50.00 psf*
Base width: 5.0 ft	

Earth Pressures:

$$k_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\,\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\,\sin(\phi-\beta)}{\sin(\alpha-\delta)\,\sin(\alpha+\beta)}}\right]^2}$$



Internal		***External***	
ϕ	= 30 deg	ϕ	= 30 deg
α	= 90.00 deg	α	= 90.00 deg
β	= 26.00 deg	β	= 26.00 deg
δ	= 26.00 deg	δ	= 26.00 deg
H	= 4.00 ft		
ka	= 0.519	ka	= 0.514

Hinge Height: Hinge Ht= Not applicable

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	*RFcr*	*RFd*	*RFid*	*LTDS*	*FS*	*Tal*	*Ci*	*Cds*
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Connection Parameters: Mirafi XT Geogrids

	Frictional 1	***Break Pt***	***Frictional 2***
5XT	*Tcl= Ntan(27.00) + 1122*	*1723*	*Tcl= Ntan(0.00) +2000*

Unit Shear Data

Shear = N tan(40.00)

Inter-Unit ShearShear = N tan(26.90) + 769.00

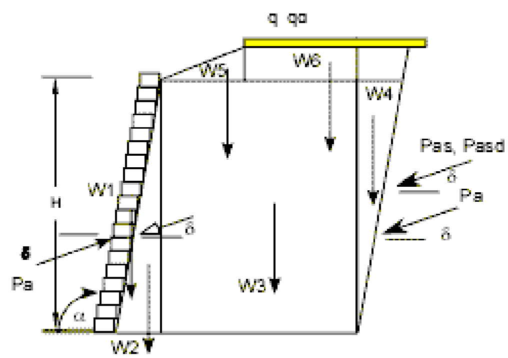
Calculated Reactions

For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 5.00 ft

$$Pa := 0.5H\cdot\left(\gamma\cdot H\cdot ka - 2c\cdot\sqrt{ka}\right) \qquad P_q := q\cdot H\cdot ka$$

$$Pa_h := Pa\cdot\cos(\delta) \qquad Pq_h := P_q\cdot\cos(\delta)$$

$$Pa_v := Pa\cdot\sin(\delta) \qquad Pq_v := P_q\cdot\sin(\delta)$$



Reactions are:

Area	**Force**	**Arm-x**	**Arm-y**	**Moment**
W1	*480.00*	*[0.500]*	*2.000*	*240.00*
W3	*1920.00*	*[3.000]*	*2.000*	*5760.00*
W5	*468.22*	*[3.667]*	*4.650*	*1716.82*
Pa_h	*981.47*	*5.000*	*[1.984]*	*-1946.88*
Pa_v	*478.69*	*[5.000]*	*1.984*	*2393.47*
Pql_h	*8.42*	*5.000*	*[2.975]*	*-25.06*
Pql_v	*4.11*	*[5.000]*	*2.975*	*20.54*
Sum V =	*3351.02*		*Sum Mr =*	*10130.82*
Sum H =	*989.89*		*Sum Mo =*	*-1971.94*

Calculate Sliding at Base

For Sliding, Vertical Force = W1+W2+W3+W4+W5+W6+qd	= 3351
The resisting force within the rein. mass , Rf_1	= N tan(30)
	= 1935
The resisting force at the foundation, Rf_2	= N tan(30.00)
	= 1935
The driving forces, Df, are the sum of the external earth pressures:	
Pa_h + Pql_h + Pqd_h	= 990
the Factor of Safety for Sliding is Rf_2/Df	= 1.95

Calculate Overturning:

Overturning moment: Mo = Sum Mo	= -1972
Resisting moment: Mr = Sum Mr	= 10110
Factor of Safety of Overturning: Mr/Mo	= 5.13

Calculate eccentricity at base: with Surcharge / without Surcharge
Sum Moments = 8138 / 8138
Sum Vertical = 3347/3347
Base Length = 5.00
e = 0.068 / 0.068

Calculate Ultimate Bearing based on shear:
where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 9849 psf

Equivalent footing width, B' = L -2e = 4.86 / 4.86
Bearing pressure = sumV/B' = 688 psf / 688 psf [bearing is greatest without liveload]
Factor of Safety for bearing = Qult/bearing= 14.31

Calculate Tensions in Reinforcing:
The tensions in the reinforcing layer, and the assumed load at the connection,
is the vertical area supported by each respective layer, Sv.Column [7] is '2c sqrt(ka)'.

Table of Results ppf

[1] Layer	[2] Depth zi	[3] h1	[4] ka/rho	[5] Pa	[6] (Pas+Pasd)	[7] c	[8] (5+6)cos(d)-7	[9] Ti	[10] Tcl	[11] Tsc
2	1.33	1.17	0.519/42	170	0	0	152	152	802	N/A
1	3.33	3.17	0.519/42	329	0	0	296	296	884	N/A

Calculate sliding on the reinforcing:

The shear value is the lessor of base-shear or inter-unit shear.

[1] Layer	[2] Depth zi	[3] N	[4] Li	[5] Cds	[6] τ	[7] RF	[8] ka	[9] Pa	[10] Pas+Pasd	[11] DF	[12] FS
2	*1.33*	*1256*	*4.00*	*0.80*	*850*	*1430*	*0.519*	*336*	*0*	*302*	*4.73*
1	*3.33*	*2453*	*4.00*	*0.80*	*972*	*2105*	*0.519*	*870*	*8*	*789*	*2.67*

Calculate pullout of each layer

The FoS (R*/S*) of pullout is calculated as the individual layer pullout (Rf) divided by the tension (Df) in that layer.
The angle of the failure plane is: 30.00 degrees from vertical.

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]
Layer	Depth zi	Le	SumV	Ci	POi	Ti	FS_PO
2	*1.33*	*2.46*	*793*	*0.80*	*732*	*152*	*4.80*
1	*3.33*	*3.62*	*1910*	*0.80*	*1764*	*296*	*5.97*

Check Shear & Bending at each layer

Bending on the top layer is the FOS of overturning of the Units
(Most surcharge loads need to be moved back from the face.)

[1]	*[2]*	*[3]*	*[4]*	*[5]*	*[6]*	*[7]*	*[8]*	*[9]*	
Layer	*Depth zi*	*Si*	*DM*	*Pv*	*RM*	*FS_b*	*DS*	*RS*	*FS_Sh*
2	*1.33*	*1.33*	*22*	*160*	*80*	*3.62*	*50*	*850*	*17.07*
Seismic	*1.33*	*1.33*	*81*	*160*	*80*	*0.99*	*129*	*850*	*6.59*
1	*3.33*	*2.00*	*75*	*280*	*247*	*3.30*	*159*	*972*	*6.12*
Seismic	*3.33*	*2.00*	*123*	*280*	*247*	*2.00*	*261*	*972*	*3.73*

EXTERNAL STABILITY

Horizontal Acceleration = 0.20g
Vertical Acceleration = 0.00g
Am= (1.45-A)A = 0.250
Kh(ext) = Am/2 = 0.125
Kh(int) = Am = 0.250
Inertia Force of the Face:
W1s = H x Wu x gamma = 480.00 ppf

Inertia Forces of the soil mass:
W2s = H x (H2/2 - face depth) * gamma
= 4.00 x 1.32 x 120.00
= 634.81 ppf
W3s = 1/2 x sqr(H2/2 - 1 ft) x tan(beta) x gamma
= 51.18 ppf
Pif = W1 * kh(ext) = 480.00 x 0.125 = 60.00
Pir = W2s * kh(ext) = 634.81 x 0.125 = 79.35
Pis = W3s * kh(ext) = 51.18 x 0.125 = 6.40

Seismic Thrust , Pae
D_Kae = Kae - Ka = 1.022 - 0.514 = 0.508
Pae = 0.5 x gamma x sqr(H2) x D_Kae = 0.5 x 120.00 x sqr(4.65) x 0.508 = 657.13
Pae_h/2 = Pae x cos(delta)/2 = 295.31
Pae_v/2 = Pae x sin(delta)/2 = 144.03

Calculated Reactions
For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 5.00 ft

Reactions for Seismic Calculations

Area	*Force*	*Arm-x*	*Arm-y*	*Moment*
W1	*480.00*	*[0.500]*	*2.000*	*240.00*
W3	*1920.00*	*[3.000]*	*2.000*	*5760.00*
W5	*468.22*	*[3.667]*	*4.650*	*1716.82*
Pa_h	*981.47*	*5.000*	*[1.984]*	*-1946.88*
Pa_v	*478.69*	*[5.000]*	*1.984*	*2393.47*
Pir	*79.35*	*1.661*	*[2.000]*	*-158.70*
P_if	*60.00*	*0.500*	*[2.000]*	*-120.00*
P_is	*6.40*	*1.882*	*[4.215]*	*-26.97*
Pae_h/2	*295.31*	*2.323*	*[2.787]*	*-823.04*
Pae_v/2	*144.03*	*[2.323]*	*2.787*	*334.52*
Sum V =	*3490.95*		*Sum Mr =*	*10444.80*
Sum H =	*1422.53*		*Sum Mo =*	*-3075.59*

Sliding Calculations

Pa_h	= 981.47 ppf
Pae_h/2	= 295.31 ppf
PIR	= 145.75 ppf
Resisting Forces, RF	= (W1 + W2 + W3 + W4 + W5 + W6 + Pav +Pae_v)tan(phi)
Foundation fill	= 3490.95 x tan(30.00) =2015.50
FS	= RF/(Pa_h + Pae_h/2 + P_ir)
	= 1.42

Overturning Calculations

Overturning moment: Mo = Sum Mo	= -3076
Resisting Moments Mr = Sum Mr	= 10445
Factor of Safety of Overturning = Mr/Mo	= 3.40

Calculate eccentricity at base:

Sum Moments	= 7369
Sum Vertical	= 3491
Base Length	= 5.00
e	= 0.389

Calculate Ultimate Bearing based on shear:

where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 8987 psf

Equivalent footing width, B' = L -2e	= 4.22
Bearing pressure = sumV/B'	= 827 psf
Factor of Safety for bearing = Qult/bearing	= 10.87

INTERNAL STABILITY

kh(int) = (1.45-A) A
= (1.45 - 0.20) 0.20 = 0.250

Inertia Forces

W1 = 1.00 x 4.00 x 120.00 x kh_int) = 120.00 ppf
Wedge = Wedge x kh_int [for failure plane angle of 60.00deg.]
= 771.51 x 0.25 = 192.88 ppf
Dead Load = = 0.00 ppf

Total Additional Internal Dynamic Loading
192.88 + 120.00 + 0.00 = 312.88 ppf

Tension in Reinforcing

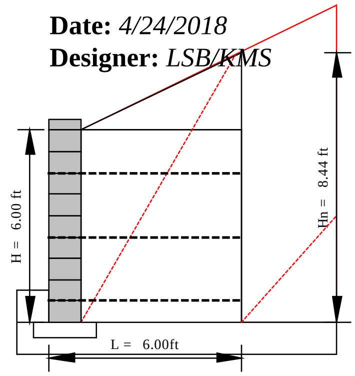
Layer	Le (ft)	Tension	Dyn Tension	Total Tension(ppf)	FoS Pullout
2	2.46	152.49	126.71	279.19	2.10
1	3.62	295.64	186.17	481.81	2.93

RETAINING WALL DESIGN

KeyWall_2012 Version 3.7.2 Build 10

Project: *5660 East Mercer Way Walls*
Project No: *1024718*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Date: *4/24/2018*
Designer: *LSB/KMS*



Design Parameters

Soil Parameters:	**ϕ deg**	**c psf**	**γ pcf**
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Reinforced Fill Type: *Sand, Silt or Clay*
Unit Fill: *Crushed Stone, 1 inch minus*

Seismic Design **A=***0.20 g*, **Kh(Ext)=***0.125* , **Kh(Int)=***0.250* , **Kv=***0.000*

Minimum Design Factors of Safety (seismic are 75% of static)

sliding:	1.50/1.13	pullout:	1.50/1.13	uncertainties:	1.50/1.13
overturning:	2.00/1.50	shear:	1.50/1.13	connection:	1.50/1.13
bearing:	2.00/1.50	bending:	1.50/1.13		

Design Preferences

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	*RFcr*	*RFd*	*RFid*	*LTDS*	*FS*	*Tal*	*Ci*	*Cds*
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Analysis: ***Case: Case 1***

6.0 - foot wall

Unit Type:	*Compac / 120.00 pcf*	*Wall Batter: 0.00 deg (Hinge Ht N/A)*
Leveling Pad:	*Crushed Stone*	
Wall Ht:	*6.00 ft*	*embedment: 1.00 ft*
BackSlope:	*26.00 deg. slope,*	*15.00 ft long*
Surcharge:	*LL: 50 psf uniform surcharge*	*DL: 0 psf uniform surcharge*
	Load Width: 100.00 ft	*Load Width: 100.00 ft*

Results:	***Sliding***	***Overturning***	***Bearing***	***Shear***	***Bending***
Factors of Safety:	*1.82/1.22*	*4.01/2.35*	*8.68/5.14*	*4.04/2.58*	*3.11/0.99<<*

Calculated Bearing Pressure: *1096 / 1096 / 1521 psf*
Eccentricity at base: 0.28 ft/0.92 ft
Reinforcing: (ft & lbs/ft)

Layer	**Height**	**Length**	**Calc. Tension**	**Reinf. Type**	**Allow Ten Tal**	**Pk Conn Tcl**	**Pullout FS**
3	*4.67*	*6.0*	*152 / 289*	*5XT*	*1717/3617 ok*	*802/1070 ok*	*5.38/2.27 ok*
2	*2.67*	*6.0*	*373 / 578*	*5XT*	*1717/3617 ok*	*884/1178 ok*	*5.06/2.62 ok*
1	*0.67*	*6.0*	*482 / 755*	*5XT*	*1717/3617 ok*	*965/1287 ok*	*7.05/3.60 ok*

Reinforcing Quantities (no waste included):
5XT 2.00 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

DETAILED CALCULATIONS

Project: *5660 East Mercer Way Walls* **Date:** *4/24/2018*
Project No: *1024718* **Designer:** *LSB/KMS*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Soil Parameters:	**ϕ deg**	**c psf**	**γ pcf**
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Leveling Pad: ***Crushed Stone***

Modular Concrete Unit: *Compac*
Depth: *1.00 ft* **In-Place Wt:** *120 pcf*

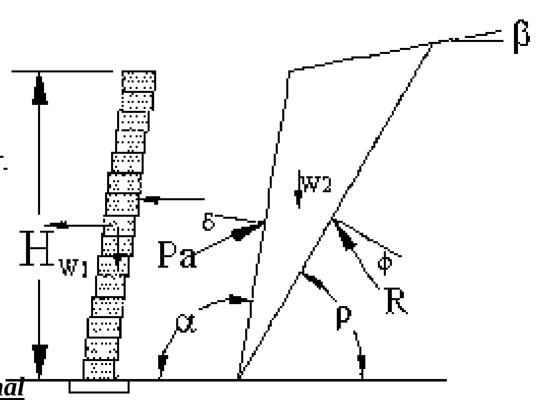
Geometry

Internal Stability
- *(Sloping geometry)*
- *Height: 6.00 ft*
- *BackSlope:*
  - *Angle: 26.0 deg*
  - *Height: 7.32 ft*
- *Batter: 0.00deg*
- *Surcharge:*
  - *Dead Load: 0.00 psf*
  - *Live Load: 0 psf*
- *Base width: 6.0 ft*

External Stability
- *(Broken geometry)*
- *Height: 8.44 ft*
  - *Angle: 26.0 deg*
  - *Height: 4.88 ft*
- *Batter: 0.00deg*
  - *Dead Load: 0.00 psf*
  - *Live Load:50.00 psf*

Earth Pressures:

$$k_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\,\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$



Internal		***External***	
ϕ	= 30 deg	ϕ	= 30 deg
α	= 90.00 deg	α	= 90.00 deg
β	= 26.00 deg	β	= 26.00 deg
δ	= 26.00 deg	δ	= 26.00 deg
H	= 6.00 ft		
ka	= 0.517	ka	= 0.486

Hinge Height: Hinge Ht= Not applicable

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	RFcr	RFd	RFid	LTDS	FS	Tal	Ci	Cds
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Connection Parameters: Mirafi XT Geogrids

	Frictional 1	Break Pt	Frictional 2
5XT	*Tcl= Ntan(27.00) + 1122*	*1723*	*Tcl= Ntan(0.00) +2000*

Unit Shear Data

Shear = N tan(40.00)
Inter-Unit ShearShear = N tan(26.90) + 769.00

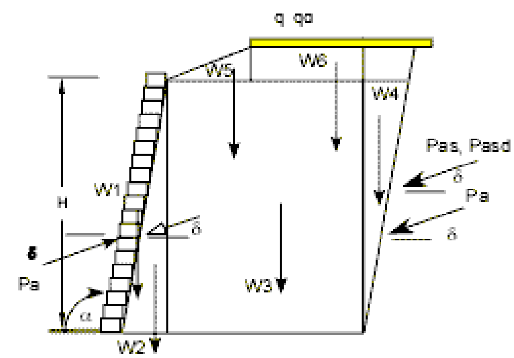
Calculated Reactions

For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 6.00 ft

$$Pa := 0.5H\cdot\left(\gamma\cdot H\cdot ka - 2c\cdot\sqrt{ka}\right) \qquad P_q := q\cdot H\cdot ka$$

$$Pa_h := Pa\cdot\cos(\delta) \qquad Pq_{lh} := P_q\cdot\cos(\delta)$$

$$Pa_v := Pa\cdot\sin(\delta) \qquad Pq_{lv} := P_q\cdot\sin(\delta)$$



Reactions are:

Area	Force	Arm-x	Arm-y	Moment
W1	*720.00*	*[0.500]*	*3.000*	*360.00*
W3	*3600.00*	*[3.500]*	*3.000*	*12600.00*
W5	*731.60*	*[4.333]*	*6.813*	*3170.26*
Pa_h	*1865.97*	*6.000*	*[2.813]*	*-5248.75*
Pa_v	*910.09*	*[6.000]*	*2.813*	*5460.55*
Pql_h	*30.58*	*6.000*	*[4.219]*	*-129.03*
Pql_v	*14.91*	*[6.000]*	*4.219*	*89.49*
Sum V =	*5976.61*		*Sum Mr =*	*21680.30*
Sum H =	*1896.55*		*Sum Mo =*	*-5377.78*

Calculate Sliding at Base

For Sliding, Vertical Force = W1+W2+W3+W4+W5+W6+qd	= 5977
The resisting force within the rein. mass , Rf_1	= N tan(30)
	= 3451
The resisting force at the foundation, Rf_2	= N tan(30.00)
	= 3451
The driving forces, Df, are the sum of the external earth pressures:	
Pa_h + Pql_h + Pqd_h	= 1897
the Factor of Safety for Sliding is Rf_2/Df	= 1.82

Calculate Overturning:

Overturning moment: Mo = Sum Mo	= -5378
Resisting moment: Mr = Sum Mr	= 21591
Factor of Safety of Overturning: Mr/Mo	= 4.01

Calculate eccentricity at base: with Surcharge / without Surcharge
Sum Moments = 16213 / 16213
Sum Vertical = 5962/5962
Base Length = 6.00
e = 0.280 / 0.280

Calculate Ultimate Bearing based on shear:
where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 9519 psf

Equivalent footing width, B' = L -2e = 5.44 / 5.44
Bearing pressure = sumV/B' = 1096 psf / 1096 psf [bearing is greatest without liveload]
Factor of Safety for bearing = Qult/bearing= 8.68

Calculate Tensions in Reinforcing:
The tensions in the reinforcing layer, and the assumed load at the connection,
is the vertical area supported by each respective layer, Sv.Column [7] is '2c sqrt(ka)'.

Table of Results ppf

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]
Layer	Depth zi	h1	ka/rho	Pa	(Pas+Pasd)	c	(5+6)cos(d)-7	Ti	Tcl	Tsc
3	1.33	1.17	0.519/42	170	0	0	152	152	802	N/A
2	3.33	3.33	0.519/42	415	0	0	373	373	884	N/A
1	5.33	5.17	0.519/42	537	0	0	482	482	965	N/A

Calculate sliding on the reinforcing:

The shear value is the lessor of base-shear or inter-unit shear.

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]
Layer	Depth zi	N	Li	Cds	τ	RF	ka	Pa	Pas+Pasd	DF	FS
3	*1.33*	*1726*	*5.00*	*0.80*	*850*	*1647*	*0.519*	*443*	*0*	*398*	*4.13*
2	*3.33*	*3184*	*5.00*	*0.80*	*972*	*2443*	*0.510*	*1019*	*14*	*928*	*2.63*
1	*5.33*	*4725*	*5.00*	*0.80*	*1094*	*3276*	*0.490*	*1776*	*35*	*1627*	*2.01*

Calculate pullout of each layer

The FoS (R*/S*) of pullout is calculated as the individual layer pullout (Rf) divided by the tension (Df) in that layer.
The angle of the failure plane is: 30.00 degrees from vertical.

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]
Layer	Depth zi	Le	SumV	Ci	POi	Ti	FS_PO
3	*1.33*	*2.31*	*888*	*0.80*	*820*	*152*	*5.38*
2	*3.33*	*3.46*	*2046*	*0.80*	*1890*	*373*	*5.06*
1	*5.33*	*4.62*	*3681*	*0.80*	*3400*	*482*	*7.05*

Check Shear & Bending at each layer

Bending on the top layer is the FOS of overturning of the Units
(Most surcharge loads need to be moved back from the face.)

[1]	*[2]*	*[3]*	*[4]*	*[5]*	*[6]*	*[7]*	*[8]*	*[9]*	
Layer	*Depth zi*	*Si*	*DM*	*Pv*	*RM*	*FS_b*	*DS*	*RS*	*FS_Sh*
3	*1.33*	*1.33*	*22*	*160*	*80*	*3.62*	*50*	*850*	*17.07*
Seismic	*1.33*	*1.33*	*81*	*160*	*80*	*0.99*	*129*	*850*	*6.59*
2	*3.33*	*2.00*	*75*	*280*	*247*	*3.30*	*159*	*972*	*6.12*
Seismic	*3.33*	*2.00*	*120*	*280*	*247*	*2.05*	*254*	*972*	*3.82*
1	*5.33*	*2.00*	*131*	*520*	*407*	*3.11*	*271*	*1094*	*4.04*
Seismic	*5.33*	*2.00*	*204*	*520*	*407*	*1.99*	*423*	*1094*	*2.58*

EXTERNAL STABILITY

Horizontal Acceleration = 0.20g
Vertical Acceleration = 0.00g
Am= (1.45-A)A = 0.250
Kh(ext) = Am/2 = 0.125
Kh(int) = Am = 0.250
Inertia Force of the Face:
W1s = H x Wu x gamma = 720.00 ppf

Inertia Forces of the soil mass:
W2s = H x (H2/2 - face depth) * gamma
= 6.00 x 2.65 x 120.00
= 1904.43 ppf
W3s = 1/2 x sqr(H2/2 - 1 ft) x tan(beta) x gamma
= 204.74 ppf
Pif = W1 * kh(ext) = 720.00 x 0.125 = 90.00
Pir = W2s * kh(ext) = 1904.43 x 0.125 = 238.05
Pis = W3s * kh(ext) = 204.74 x 0.125 = 25.59

Seismic Thrust , Pae
D_Kae = Kae - Ka = 1.022 - 0.486 = 0.536
Pae = 0.5 x gamma x sqr(H2) x D_Kae = 0.5 x 120.00 x sqr(7.29) x 0.536 = 1707.94
Pae_h/2 = Pae x cos(delta)/2 = 767.54
Pae_v/2 = Pae x sin(delta)/2 = 374.36

Calculated Reactions
For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 6.00 ft

Reactions for Seismic Calculations

Area	*Force*	*Arm-x*	*Arm-y*	*Moment*
W1	*720.00*	*[0.500]*	*3.000*	*360.00*
W3	*3600.00*	*[3.500]*	*3.000*	*12600.00*
W5	*731.60*	*[4.333]*	*6.813*	*3170.26*
Pa_h	*1865.97*	*6.000*	*[2.813]*	*-5248.75*
Pa_v	*910.09*	*[6.000]*	*2.813*	*5460.55*
Pir	*238.05*	*2.323*	*[3.000]*	*-714.16*
P_if	*90.00*	*0.500*	*[3.000]*	*-270.00*
P_is	*25.59*	*2.763*	*[6.430]*	*-164.56*
Pae_h/2	*767.54*	*3.645*	*[4.374]*	*-3357.26*
Pae_v/2	*374.36*	*[3.645]*	*4.374*	*1364.54*
Sum V =	*6336.05*		*Sum Mr =*	*22955.35*
Sum H =	*2987.15*		*Sum Mo =*	*-9754.73*

Sliding Calculations

Pa_h	= 1865.97 ppf
Pae_h/2	= 767.54 ppf
PIR	= 353.65 ppf
Resisting Forces, RF	= (W1 + W2 + W3 + W4 + W5 + W6 + Pav +Pae_v)tan(phi)
Foundation fill	= 6336.05 x tan(30.00) =3658.12
FS	= RF/(Pa_h + Pae_h/2 + P_ir)
	= 1.22

Overturning Calculations

Overturning moment: Mo = Sum Mo	= -9755
Resisting Moments Mr = Sum Mr	= 22955
Factor of Safety of Overturning = Mr/Mo	= 2.35

Calculate eccentricity at base:

Sum Moments	= 13201
Sum Vertical	= 6336
Base Length	= 6.00
e	= 0.917

Calculate Ultimate Bearing based on shear:

where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 7809 psf

Equivalent footing width, B' = L -2e	= 4.17
Bearing pressure = sumV/B'	= 1521 psf
Factor of Safety for bearing = Qult/bearing	= 5.14

INTERNAL STABILITY

kh(int) = (1.45-A) A
= (1.45 - 0.20) 0.20 = 0.250

Inertia Forces
W1 = 1.00 x 6.00 x 120.00 x kh_int) = 180.00 ppf
Wedge = Wedge x kh_int [for failure plane angle of 60.00deg.]
= 1735.89 x 0.25 = 433.97 ppf
Dead Load = = 0.00 ppf

Total Additional Internal Dynamic Loading
433.97 + 180.00 + 0.00 = 613.97 ppf

Tension in Reinforcing

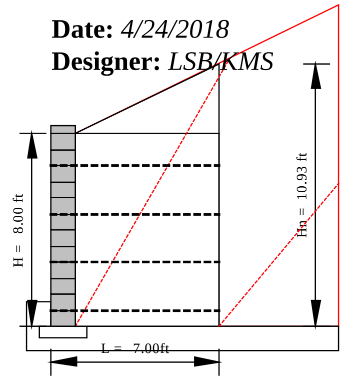
Layer	Le (ft)	Tension	Dyn Tension	Total Tension(ppf)	FoS Pullout
3	2.31	152.49	136.37	288.85	2.27
2	3.46	373.43	204.66	578.09	2.62
1	4.62	482.25	272.95	755.20	3.60

RETAINING WALL DESIGN

KeyWall_2012 Version 3.7.2 Build 10

Project: *5660 East Mercer Way Walls*
Project No: *1024718*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Date: *4/24/2018*
Designer: *LSB/KMS*



Design Parameters

Soil Parameters:	ϕ deg	c psf	γ pcf
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Reinforced Fill Type: *Sand, Silt or Clay*
Unit Fill: *Crushed Stone, 1 inch minus*

Seismic Design A=*0.20 g,* **Kh(Ext)=***0.125* , **Kh(Int)=***0.250* , **Kv=***0.000*

Minimum Design Factors of Safety (seismic are 75% of static)

sliding:	1.50/1.13	pullout:	1.50/1.13	uncertainties:	1.50/1.13
overturning:	2.00/1.50	shear:	1.50/1.13	connection:	1.50/1.13
bearing:	2.00/1.50	bending:	1.50/1.13		

Design Preferences

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	RFcr	RFd	RFid	LTDS	FS	Tal	Ci	Cds
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Analysis: ***Case: Case 1***

8.0 - foot wall

Unit Type:	*Compac / 120.00 pcf*	*Wall Batter: 0.00 deg (Hinge Ht N/A)*
Leveling Pad:	*Crushed Stone*	
Wall Ht:	*8.00 ft*	*embedment: 1.00 ft*
BackSlope:	*26.00 deg. slope,*	*15.00 ft long*
Surcharge:	*LL: 50 psf uniform surcharge*	*DL: 0 psf uniform surcharge*
	Load Width: 100.00 ft	*Load Width: 100.00 ft*

Results:	Sliding	Overturning	Bearing	Shear	Bending
Factors of Safety:	*1.70/1.30*	*3.30/2.35*	*6.41/4.51*	*3.19/2.07*	*3.04/1.32*

Calculated Bearing Pressure: *1554 / 1554 / 1913 psf*
Eccentricity at base: 0.61 ft/1.11 ft
Reinforcing: (ft & lbs/ft)

Layer	**Height**	**Length**	**Calc. Tension**	**Reinf. Type**	**Allow Ten Tal**	**Pk Conn Tcl**	**Pullout FS**
4	*6.67*	*7.0*	*153 / 293*	*5XT*	*1717/3617 ok*	*802/1070 ok*	*5.81/2.43 ok*
3	*4.67*	*7.0*	*376 / 591*	*5XT*	*1717/3617 ok*	*884/1178 ok*	*5.32/2.71 ok*
2	*2.67*	*7.0*	*601 / 891*	*5XT*	*1717/3617 ok*	*965/1287 ok*	*5.90/3.18 ok*
1	*0.67*	*7.0*	*666 / 1031*	*5XT*	*1717/3617 ok*	*1047/1396 ok*	*8.31/4.29 ok*

Reinforcing Quantities (no waste included):
5XT 3.11 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

DETAILED CALCULATIONS

Project: *5660 East Mercer Way Walls* **Date:** *4/24/2018*
Project No: *1024718* **Designer:** *LSB/KMS*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Soil Parameters:	**φ deg**	**c psf**	**γ pcf**
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Leveling Pad: ***Crushed Stone***

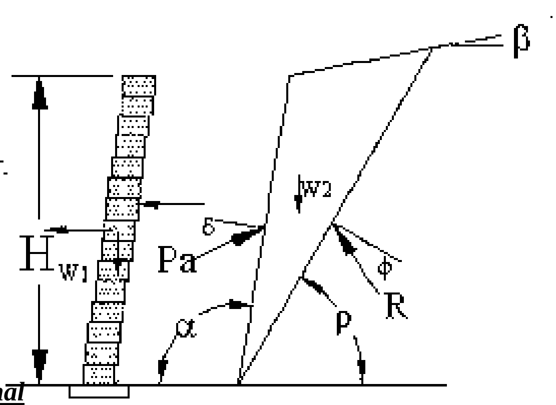
Modular Concrete Unit: *Compac*
Depth: *1.00 ft* **In-Place Wt:** *120 pcf*

Geometry

Internal Stability	*External Stability*
(Broken geometry)	*(Broken geometry)*
Height: 8.00 ft	*Height: 10.93 ft*
BackSlope:	
Angle: 26.0 deg	*Angle: 26.0 deg*
Height: 7.32 ft	*Height: 4.39 ft*
Batter: 0.00deg	*Batter: 0.00deg*
Surcharge:	
Dead Load: 0.00 psf	*Dead Load: 0.00 psf*
Live Load: 50.00 psf	*Live Load:50.00 psf*
Base width: 7.0 ft	

Earth Pressures:

$$k_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\,\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$



Internal		***External***	
φ	= 30 deg	φ	= 30 deg
α	= 90.00 deg	α	= 90.00 deg
β	= 26.00 deg	β	= 26.00 deg
δ	= 24.57 deg	δ	= 21.89 deg
H	= 8.00 ft		
ka	= 0.507	ka	= 0.451

Hinge Height: Hinge Ht= Not applicable

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	RFcr	RFd	RFid	LTDS	FS	Tal	Ci	Cds
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Connection Parameters: Mirafi XT Geogrids

	Frictional 1	Break Pt	Frictional 2
5XT	*Tcl= Ntan(27.00) + 1122*	*1723*	*Tcl= Ntan(0.00) +2000*

Unit Shear Data

Shear = N tan(40.00)

Inter-Unit ShearShear = N tan(26.90) + 769.00

Calculated Reactions

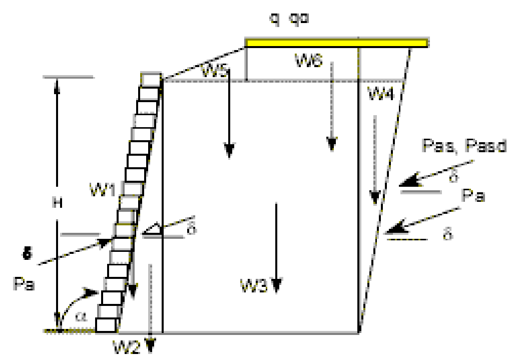
For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.

effective sliding length = 7.00 ft

$$Pa := 0.5H\cdot(\gamma\cdot H\cdot ka - 2c\cdot\sqrt{ka}) \qquad P_q := q\cdot H\cdot ka$$

$$Pa_h := Pa\cdot\cos(\delta) \qquad Pq_{lh} := P_q\cdot\cos(\delta)$$

$$Pa_v := Pa\cdot\sin(\delta) \qquad Pq_{lv} := P_q\cdot\sin(\delta)$$



Reactions are:

Area	Force	Arm-x	Arm-y	Moment
W1	*960.00*	*[0.500]*	*4.000*	*480.00*
W3	*5760.00*	*[4.000]*	*4.000*	*23040.00*
W5	*1053.50*	*[5.000]*	*8.975*	*5267.51*
Pa_h	*2997.49*	*7.000*	*[3.642]*	*-10917.25*
Pa_v	*1204.22*	*[7.000]*	*3.642*	*8429.53*
Pql_h	*68.52*	*7.000*	*[5.463]*	*-374.31*
Pql_v	*27.53*	*[7.000]*	*5.463*	*192.68*
Sum V =	*9005.25*		*Sum Mr =*	*37409.71*
Sum H =	*3066.00*		*Sum Mo =*	*-11291.56*

Calculate Sliding at Base

For Sliding, Vertical Force = W1+W2+W3+W4+W5+W6+qd	= 9005
The resisting force within the rein. mass , Rf_1	= N tan(30)
	= 5199
The resisting force at the foundation, Rf_2	= N tan(30.00)
	= 5199
The driving forces, Df, are the sum of the external earth pressures:	
Pa_h + Pql_h + Pqd_h	= 3066
the Factor of Safety for Sliding is Rf_2/Df	= 1.70

Calculate Overturning:

Overturning moment: Mo = Sum Mo	= -11292
Resisting moment: Mr = Sum Mr	= 37217
Factor of Safety of Overturning: Mr/Mo	= 3.30

Calculate eccentricity at base: with Surcharge / without Surcharge

Sum Moments = 25925 / 25925
Sum Vertical = 8978/8978
Base Length = 7.00
e = 0.612 / 0.612

Calculate Ultimate Bearing based on shear:

where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 9971 psf

Equivalent footing width, B' = L -2e = 5.78 / 5.78
Bearing pressure = sumV/B' = 1554 psf / 1554 psf [bearing is greatest without liveload]
Factor of Safety for bearing = Qult/bearing= 6.41

Calculate Tensions in Reinforcing:

The tensions in the reinforcing layer, and the assumed load at the connection,
is the vertical area supported by each respective layer, Sv.Column [7] is '2c sqrt(ka)'.

Table of Results ppf

[1] Layer	[2] Depth zi	[3] h1	[4] ka/rho	[5] Pa	[6] (Pas+Pasd)	[7] c	[8] (5+6)cos(d)-7	[9] Ti	[10] Tcl	[11] Tsc
4	1.33	1.17	0.516/43	169	0	0	153	153	802	N/A
3	3.33	3.33	0.516/43	413	0	0	376	376	884	N/A
2	5.33	5.33	0.516/42	661	0	0	601	601	965	N/A
1	7.33	7.17	0.511/44	732	0	0	666	666	1047	N/A

Calculate sliding on the reinforcing:

The shear value is the lessor of base-shear or inter-unit shear.

[1] Layer	[2] Depth zi	[3] N	[4] Li	[5] Cds	[6] τ	[7] RF	[8] ka	[9] Pa	[10] Pas+Pasd	[11] DF	[12] FS
4	*1.33*	*2223*	*6.00*	*0.80*	*850*	*1877*	*0.512*	*557*	*4*	*521*	*3.60*
3	*3.33*	*3895*	*6.00*	*0.80*	*972*	*2771*	*0.495*	*1164*	*19*	*1098*	*2.52*
2	*5.33*	*5633*	*6.00*	*0.80*	*1094*	*3696*	*0.475*	*1943*	*41*	*1841*	*2.01*
1	*7.33*	*7433*	*6.00*	*0.80*	*1215*	*4649*	*0.457*	*2886*	*63*	*2736*	*1.70*

Calculate pullout of each layer

The FoS (R*/S*) of pullout is calculated as the individual layer pullout (Rf) divided by the tension (Df) in that layer.
The angle of the failure plane is: 30.00 degrees from vertical.

[1] Layer	[2] Depth zi	[3] Le	[4] SumV	[5] Ci	[6] POi	[7] Ti	[8] FS_PO
4	*1.33*	*2.15*	*964*	*0.80*	*891*	*153*	*5.81*
3	*3.33*	*3.31*	*2163*	*0.80*	*1998*	*376*	*5.32*
2	*5.33*	*4.46*	*3839*	*0.80*	*3546*	*601*	*5.90*
1	*7.33*	*5.62*	*5990*	*0.80*	*5534*	*666*	*8.31*

Check Shear & Bending at each layer

Bending on the top layer is the FOS of overturning of the Units
(Most surcharge loads need to be moved back from the face.)

[1] Layer	*[2] Depth zi*	*[3] Si*	*[4] DM*	*[5] Pv*	*[6] RM*	*[7] FS_b*	*[8] DS*	*[9] RS*	*FS_Sh*
4	*1.33*	*1.33*	*22*	*160*	*80*	*3.59*	*50*	*850*	*16.97*
Seismic	*1.33*	*1.33*	*61*	*160*	*80*	*1.32*	*104*	*850*	*8.21*
3	*3.33*	*2.00*	*75*	*280*	*247*	*3.28*	*160*	*972*	*6.09*
Seismic	*3.33*	*2.00*	*121*	*280*	*247*	*2.03*	*256*	*972*	*3.80*
2	*5.33*	*2.00*	*131*	*520*	*407*	*3.09*	*272*	*1094*	*4.02*
Seismic	*5.33*	*2.00*	*198*	*520*	*407*	*2.06*	*408*	*1094*	*2.68*
1	*7.33*	*2.00*	*186*	*760*	*567*	*3.04*	*381*	*1215*	*3.19*
Seismic	*7.33*	*2.00*	*286*	*760*	*567*	*1.98*	*587*	*1215*	*2.07*

EXTERNAL STABILITY

Horizontal Acceleration	= 0.20g
Vertical Acceleration	= 0.00g
Am= (1.45-A)A	= 0.250
Kh(ext) = Am/2	= 0.125
Kh(int) = Am	= 0.250

Inertia Force of the Face:

W1s = H x Wu x gamma = 960.00 ppf

Inertia Forces of the soil mass:

W2s = H x (H2/2 - face depth) * gamma
= 8.00 x 3.97 x 120.00
= 3808.85 ppf

W3s = 1/2 x sqr(H2/2 - 1 ft) x tan(beta) x gamma
= 460.66 ppf

Pif = W1 * kh(ext) = 960.00 x 0.125 = 120.00

Pir = W2s * kh(ext) = 3808.85 x 0.125 = 476.11

Pis = W3s * kh(ext) = 460.66 x 0.125 = 57.58

Seismic Thrust , Pae

D_Kae = Kae - Ka = 0.597 - 0.451 = 0.146

Pae = 0.5 x gamma x sqr(H2) x D_Kae = 0.5 x 120.00 x sqr(9.94) x 0.146 = 861.98

Pae_h/2 = Pae x cos(delta)/2 = 399.93

Pae_v/2 = Pae x sin(delta)/2 = 160.67

Calculated Reactions

For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 7.00 ft

Reactions for Seismic Calculations

Area	***Force***	***Arm-x***	***Arm-y***	***Moment***
W1	*960.00*	*[0.500]*	*4.000*	*480.00*
W3	*5760.00*	*[4.000]*	*4.000*	*23040.00*
W5	*1053.50*	*[5.000]*	*8.975*	*5267.51*
Pa_h	*2997.49*	*7.000*	*[3.642]*	*-10917.25*
Pa_v	*1204.22*	*[7.000]*	*3.642*	*8429.53*
Pir	*476.11*	*2.984*	*[4.000]*	*-1904.43*
P_if	*120.00*	*0.500*	*[4.000]*	*-480.00*
P_is	*57.58*	*3.645*	*[8.645]*	*-497.80*
Pae_h/2	*399.93*	*4.968*	*[5.961]*	*-2383.98*
Pae_v/2	*160.67*	*[4.968]*	*5.961*	*798.12*
Sum V =	*9138.39*		*Sum Mr =*	*38015.16*
Sum H =	*4051.10*		*Sum Mo =*	*-16183.46*

Sliding Calculations

Pa_h	= 2997.49 ppf
Pae_h/2	= 399.93 ppf
PIR	= 653.69 ppf
Resisting Forces, RF	= (W1 + W2 + W3 + W4 + W5 + W6 + Pav +Pae_v)tan(phi)
Foundation fill	= 9138.39 x tan(30.00) =5276.05
FS	= RF/(Pa_h + Pae_h/2 + P_ir)
	= 1.30

Overturning Calculations

Overturning moment: Mo = Sum Mo	= -16183
Resisting Moments Mr = Sum Mr	= 38015
Factor of Safety of Overturning = Mr/Mo	= 2.35

Calculate eccentricity at base:

Sum Moments	= 21832
Sum Vertical	= 9138
Base Length	= 7.00
e	= 1.111

Calculate Ultimate Bearing based on shear:

where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 8631 psf

Equivalent footing width, B' = L -2e	= 4.78
Bearing pressure = sumV/B'	= 1913 psf
Factor of Safety for bearing = Qult/bearing	= 4.51

INTERNAL STABILITY

kh(int) = (1.45-A) A
= (1.45 - 0.20) 0.20 = 0.250

Inertia Forces
W1 = 1.00 x 8.00 x 120.00 x kh_int) = 240.00 ppf
Wedge = Wedge x kh_int [for failure plane angle of 60.00deg.]
= 3086.03 x 0.25 = 771.51 ppf
Dead Load = = 0.00 ppf

Total Additional Internal Dynamic Loading
771.51 + 240.00 + 0.00 = 1011.51 ppf

Tension in Reinforcing

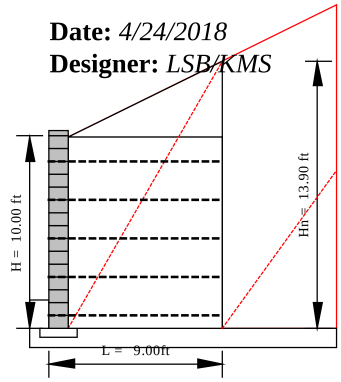
Layer	Le (ft)	Tension	Dyn Tension	Total Tension(ppf)	FoS Pullout
4	2.15	153.42	140.08	293.50	2.43
3	3.31	375.72	215.28	591.00	2.71
2	4.46	600.88	290.48	891.35	3.18
1	5.62	665.82	365.67	1031.49	4.29

RETAINING WALL DESIGN

KeyWall_2012 Version 3.7.2 Build 10

Project: *5660 East Mercer Way Walls*
Project No: *1024718*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Date: *4/24/2018*
Designer: *LSB/KMS*



Design Parameters

Soil Parameters:

	φ deg	c psf	γ pcf
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Reinforced Fill Type: *Sand, Silt or Clay*
Unit Fill: *Crushed Stone, 1 inch minus*

Seismic Design A=*0.20 g,* **Kh(Ext)=***0.125* , **Kh(Int)=***0.250* , **Kv=***0.000*

Minimum Design Factors of Safety (seismic are 75% of static)

sliding:	1.50/1.13	pullout:	1.50/1.13	uncertainties:	1.50/1.13
overturning:	2.00/1.50	shear:	1.50/1.13	connection:	1.50/1.13
bearing:	2.00/1.50	bending:	1.50/1.13		

Design Preferences

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	RFcr	RFd	RFid	LTDS	FS	Tal	Ci	Cds
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Analysis: ***Case: Case 1***

10.0 - foot wall

Unit Type:	*Compac / 120.00 pcf*	*Wall Batter: 0.00 deg (Hinge Ht N/A)*
Leveling Pad:	*Crushed Stone*	
Wall Ht:	*10.00 ft*	*embedment: 1.50 ft*
BackSlope:	*26.00 deg. slope,*	*15.00 ft long*
Surcharge:	*LL: 50 psf uniform surcharge*	*DL: 0 psf uniform surcharge*
	Load Width: 100.00 ft	*Load Width: 100.00 ft*

Results:

	Sliding	Overturning	Bearing	Shear	Bending
Factors of Safety:	*1.56/1.30*	*3.15/2.29*	*6.42/4.60*	*2.70/1.81*	*2.98/1.44*

Calculated Bearing Pressure: *1978 / 1978 / 2411 psf*
Eccentricity at base: 1.01 ft/1.61 ft
Reinforcing: (ft & lbs/ft)

Layer	Height	Length	Calc. Tension	Reinf. Type	Allow Ten Tal	Pk Conn Tcl	Pullout FS
5	*8.67*	*9.0*	*156 / 326*	*5XT*	*1717/3617 ok*	*802/1070 ok*	*9.57/3.67 ok*
4	*6.67*	*9.0*	*383 / 618*	*5XT*	*1717/3617 ok*	*884/1178 ok*	*7.48/3.70 ok*
3	*4.67*	*9.0*	*612 / 913*	*5XT*	*1717/3617 ok*	*965/1287 ok*	*7.63/4.09 ok*
2	*2.67*	*9.0*	*834 / 1200*	*5XT*	*1717/3617 ok*	*1047/1396 ok*	*8.30/4.61 ok*
1	*0.67*	*9.0*	*853 / 1283*	*5XT*	*1717/3617 ok*	*1128/1505 ok*	*>10/5.99 ok*

Reinforcing Quantities (no waste included):
5XT 5.00 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

DETAILED CALCULATIONS

Project: *5660 East Mercer Way Walls* **Date:** *4/24/2018*
Project No: *1024718* **Designer:** *LSB/KMS*
Case: *Case 1*
Design Method: *Rankine-w/Batter (modified soil interface)*

Soil Parameters:	**ϕ deg**	**c psf**	**γ pcf**
Reinforced Fill	*30*	*0*	*120*
Retained Zone	*30*	*0*	*120*
Foundation Soil	*30*	*0*	*120*

Leveling Pad: ***Crushed Stone***

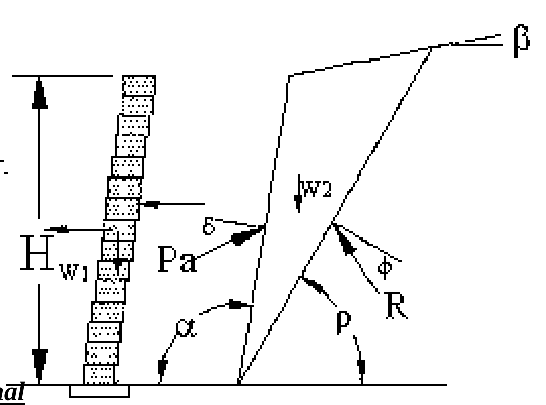
Modular Concrete Unit: *Compac*
Depth: *1.00 ft* **In-Place Wt:** *120 pcf*

Geometry

Internal Stability	*External Stability*
(Broken geometry)	*(Broken geometry)*
Height: 10.00 ft	*Height: 13.90 ft*
BackSlope:	
Angle: 26.0 deg	*Angle: 26.0 deg*
Height: 7.32 ft	*Height: 3.41 ft*
Batter: 0.00deg	*Batter: 0.00deg*
Surcharge:	
Dead Load: 0.00 psf	*Dead Load: 0.00 psf*
Live Load: 50.00 psf	*Live Load:50.00 psf*
Base width: 9.0 ft	

Earth Pressures:

$$k_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\,\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\,\sin(\phi-\beta)}{\sin(\alpha-\delta)\,\sin(\alpha+\beta)}}\right]^2}$$



Internal		***External***	
ϕ	= 30 deg	ϕ	= 30 deg
α	= 90.00 deg	α	= 90.00 deg
β	= 26.00 deg	β	= 26.00 deg
δ	= 20.09 deg	δ	= 13.80 deg
H	= 10.00 ft		
ka	= 0.490	ka	= 0.415

Hinge Height: Hinge Ht= Not applicable

Reinforcing Parameters: *Mirafi XT Geogrids*

	Tult	*RFcr*	*RFd*	*RFid*	*LTDS*	*FS*	*Tal*	*Ci*	*Cds*
5XT	*4700*	*1.58*	*1.10*	*1.05*	*2575*	*1.50*	*1717/3617*	*0.80*	*0.80*

Connection Parameters: Mirafi XT Geogrids

	Frictional 1	***Break Pt***	***Frictional 2***
5XT	*Tcl= Ntan(27.00) + 1122*	*1723*	*Tcl= Ntan(0.00) +2000*

Unit Shear Data

Shear = N tan(40.00)

Inter-Unit ShearShear = N tan(26.90) + 769.00

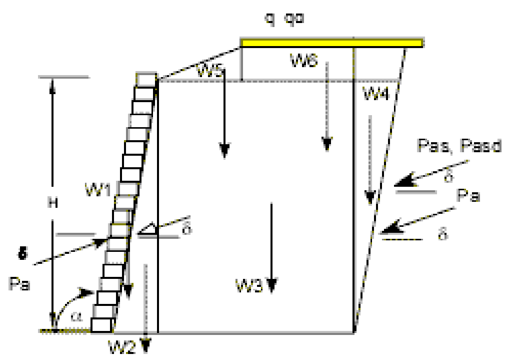
Calculated Reactions

For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 9.00 ft

$$Pa := 0.5H\cdot\left(\gamma\cdot H\cdot ka - 2c\cdot\sqrt{ka}\right) \qquad P_q := q\cdot H\cdot ka$$

$$Pa_h := Pa\cdot\cos(\delta) \qquad Pq_{lh} := P_q\cdot\cos(\delta)$$

$$Pa_v := Pa\cdot\sin(\delta) \qquad Pq_{lv} := P_q\cdot\sin(\delta)$$



Reactions are:

Area	**Force**	**Arm-x**	**Arm-y**	**Moment**
W1	*1200.00*	*[0.500]*	*5.000*	*600.00*
W3	*9600.00*	*[5.000]*	*5.000*	*48000.00*
W5	*1872.89*	*[6.333]*	*11.301*	*11861.66*
Pa_h	*4670.45*	*9.000*	*[4.634]*	*-21642.65*
Pa_v	*1147.01*	*[9.000]*	*4.634*	*10323.05*
Pql_h	*124.19*	*9.000*	*[6.951]*	*-863.25*
Pql_v	*30.50*	*[9.000]*	*6.951*	*274.50*
Sum V =	*13850.40*		*Sum Mr =*	*71059.20*
Sum H =	*4794.64*		*Sum Mo =*	*-22505.89*

Calculate Sliding at Base

For Sliding, Vertical Force = W1+W2+W3+W4+W5+W6+qd	= 13850
The resisting force within the rein. mass , Rf_1	= N tan(30)
	= 7997
The resisting force at the foundation, Rf_2	= N tan(30.00)
	= 7997
The driving forces, Df, are the sum of the external earth pressures:	
Pa_h + Pql_h + Pqd_h	= 4795
the Factor of Safety for Sliding is Rf_2/Df	= 1.67

Calculate Overturning:

Overturning moment: Mo = Sum Mo	= -22506
Resisting moment: Mr = Sum Mr	= 70785
Factor of Safety of Overturning: Mr/Mo	= 3.15

Calculate eccentricity at base: with Surcharge / without Surcharge

Sum Moments = 48279 / 48279
Sum Vertical = 13820/13820
Base Length = 9.00
e = 1.007 / 1.007

Calculate Ultimate Bearing based on shear:

where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 12704 psf

Equivalent footing width, B' = L -2e = 6.99 / 6.99
Bearing pressure = sumV/B' = 1978 psf / 1978 psf [bearing is greatest without liveload]
Factor of Safety for bearing = Qult/bearing= 6.42

Calculate Tensions in Reinforcing:

The tensions in the reinforcing layer, and the assumed load at the connection,
is the vertical area supported by each respective layer, Sv.Column [7] is '2c sqrt(ka)'.

Table of Results ppf

[1] Layer	[2] Depth zi	[3] h1	[4] ka/rho	[5] Pa	[6] (Pas+Pasd)	[7] c	[8] (5+6)cos(d)-7	[9] Ti	[10] Tcl	[11] Tsc
5	1.33	1.17	0.510/43	166	0	0	156	156	802	N/A
4	3.33	3.33	0.510/43	408	0	0	383	383	884	N/A
3	5.33	5.33	0.509/44	652	0	0	612	612	965	N/A
2	7.33	7.33	0.504/46	888	0	0	834	834	1047	N/A
1	9.33	9.17	0.494/46	906	2	0	853	853	1128	N/A

Calculate sliding on the reinforcing:

The shear value is the lessor of base-shear or inter-unit shear.

[1] Layer	[2] Depth zi	[3] N	[4] Li	[5] Cds	[6] τ	[7] RF	[8] ka	[9] Pa	[10] Pas+Pasd	[11] DF	[12] FS
5	*1.33*	*3359*	*8.00*	*0.80*	*850*	*2401*	*0.506*	*832*	*30*	*837*	*2.87*
4	*3.33*	*5432*	*8.00*	*0.80*	*972*	*3481*	*0.468*	*1471*	*37*	*1464*	*2.38*
3	*5.33*	*7555*	*8.00*	*0.80*	*1094*	*4583*	*0.449*	*2296*	*60*	*2289*	*2.00*
2	*7.33*	*9714*	*8.00*	*0.80*	*1215*	*5702*	*0.431*	*3265*	*94*	*3262*	*1.75*
1	*9.33*	*11910*	*8.00*	*0.80*	*1337*	*6838*	*0.419*	*4401*	*117*	*4388*	*1.56*

Calculate pullout of each layer

The FoS (R*/S*) of pullout is calculated as the individual
layer pullout (Rf) divided by the tension (Df) in that layer.
The angle of the failure plane is: 30.00 degrees from vertical.

[1] Layer	[2] Depth zi	[3] Le	[4] SumV	[5] Ci	[6] POi	[7] Ti	[8] FS_PO
5	*1.33*	*3.00*	*1620*	*0.80*	*1496*	*156*	*9.57*
4	*3.33*	*4.15*	*3100*	*0.80*	*2863*	*383*	*7.48*
3	*5.33*	*5.31*	*5056*	*0.80*	*4671*	*612*	*7.63*
2	*7.33*	*6.46*	*7489*	*0.80*	*6918*	*834*	*8.30*
1	*9.33*	*7.62*	*10397*	*0.80*	*9605*	*853*	*11.25*

Check Shear & Bending at each layer

Bending on the top layer is the FOS of overturning of the Units
(Most surcharge loads need to be moved back from the face.)

[1] Layer	*[2] Depth zi*	*[3] Si*	*[4] DM*	*[5] Pv*	*[6] RM*	*[7] FS_b*	*[8] DS*	*[9] RS*	*FS_Sh*
5	*1.33*	*1.33*	*23*	*160*	*80*	*3.53*	*51*	*850*	*16.65*
Seismic	*1.33*	*1.33*	*55*	*160*	*80*	*1.44*	*97*	*850*	*8.78*
4	*3.33*	*2.00*	*77*	*280*	*247*	*3.22*	*163*	*972*	*5.97*
Seismic	*3.33*	*2.00*	*129*	*280*	*247*	*1.92*	*270*	*972*	*3.60*
3	*5.33*	*2.00*	*134*	*520*	*407*	*3.04*	*277*	*1094*	*3.94*
Seismic	*5.33*	*2.00*	*204*	*520*	*407*	*2.00*	*420*	*1094*	*2.61*
2	*7.33*	*2.00*	*190*	*760*	*567*	*2.98*	*389*	*1215*	*3.12*
Seismic	*7.33*	*2.00*	*276*	*760*	*567*	*2.05*	*564*	*1215*	*2.15*
1	*9.33*	*2.00*	*243*	*1000*	*727*	*2.99*	*495*	*1337*	*2.70*
Seismic	*9.33*	*2.00*	*362*	*1000*	*727*	*2.01*	*739*	*1337*	*1.81*

EXTERNAL STABILITY

Horizontal Acceleration	= 0.20g
Vertical Acceleration	= 0.00g
Am= (1.45-A)A	= 0.250
Kh(ext) = Am/2	= 0.125
Kh(int) = Am	= 0.250
Inertia Force of the Face:	
W1s	= H x Wu x gamma = 1200.00 ppf

Inertia Forces of the soil mass:	
W2s	= H x (H2/2 - face depth) * gamma
	= 10.00 x 5.29 x 120.00
	= 6348.08 ppf
W3s	= 1/2 x sqr(H2/2 - 1 ft) x tan(beta) x gamma
	= 818.95 ppf
Pif	= W1 * kh(ext) = 1200.00 x 0.125 = 150.00
Pir	= W2s * kh(ext) = 6348.08 x 0.125 = 793.51
Pis	= W3s * kh(ext) = 818.95 x 0.125 = 102.37

Seismic Thrust , Pae	
D_Kae	= Kae - Ka = 0.523 - 0.415 = 0.108
Pae	= 0.5 x gamma x sqr(H2) x D_Kae = 0.5 x 120.00 x sqr(12.58) x 0.108 = 1028.41
Pae_h/2	= Pae x cos(delta)/2 = 499.37
Pae_v/2	= Pae x sin(delta)/2 = 122.64

Calculated Reactions

For the "modified" design method, the back of the mass assumed to be vertical for calculation of resisting forces.
effective sliding length = 9.00 ft

Reactions for Seismic Calculations

Area	***Force***	***Arm-x***	***Arm-y***	***Moment***
W1	*1200.00*	*[0.500]*	*5.000*	*600.00*
W3	*9600.00*	*[5.000]*	*5.000*	*48000.00*
W5	*1872.89*	*[6.333]*	*11.301*	*11861.66*
Pa_h	*4670.45*	*9.000*	*[4.634]*	*-21642.65*
Pa_v	*1147.01*	*[9.000]*	*4.634*	*10323.05*
Pir	*793.51*	*3.645*	*[5.000]*	*-3967.55*
P_if	*150.00*	*0.500*	*[5.000]*	*-750.00*
P_is	*102.37*	*4.527*	*[10.860]*	*-1111.73*
Pae_h/2	*499.37*	*6.290*	*[7.548]*	*-3769.25*
Pae_v/2	*122.64*	*[6.290]*	*7.548*	*771.40*
Sum V =	*13942.54*		*Sum Mr =*	*71556.11*
Sum H =	*6215.69*		*Sum Mo =*	*-31241.18*

Sliding Calculations

Pa_h	= 4670.45 ppf
Pae_h/2	= 499.37 ppf
PIR	= 1045.88 ppf
Resisting Forces, RF	= (W1 + W2 + W3 + W4 + W5 + W6 + Pav +Pae_v)tan(phi)
Foundation fill	= 13942.54 x tan(30.00) =8049.73
FS	= RF/(Pa_h + Pae_h/2 + P_ir)
	= 1.30

Overturning Calculations

Overturning moment: Mo = Sum Mo	= -31241
Resisting Moments Mr = Sum Mr	= 71556
Factor of Safety of Overturning = Mr/Mo	= 2.29

Calculate eccentricity at base:

Sum Moments	= 40315
Sum Vertical	= 13943
Base Length	= 9.00
e	= 1.608

Calculate Ultimate Bearing based on shear:

where:

Nq = 18.40
Nc = 30.14
Ng = 22.40 (ref. Vesic(1973, 1975) eqns)
Qult = 11085 psf

Equivalent footing width, B' = L -2e	= 5.78
Bearing pressure = sumV/B'	= 2411 psf
Factor of Safety for bearing = Qult/bearing	= 4.60

INTERNAL STABILITY

kh(int) = (1.45-A) A
= (1.45 - 0.20) 0.20 = 0.250

Inertia Forces
W1 = 1.00 x 10.00 x 120.00 x kh_int) = 300.00 ppf
Wedge = Wedge x kh_int [for failure plane angle of 60.00deg.]
= 4821.92 x 0.25 = 1205.48 ppf
Dead Load = = 0.00 ppf

Total Additional Internal Dynamic Loading
1205.48 + 300.00 + 0.00 = 1505.48 ppf

Tension in Reinforcing

Layer	Le (ft)	Tension	Dyn Tension	Total Tension(ppf)	FoS Pullout
5	3.00	156.35	170.04	326.39	3.67
4	4.15	382.91	235.57	618.48	3.70
3	5.31	611.91	301.10	913.00	4.09
2	6.46	833.67	366.62	1200.29	4.61
1	7.62	851.26	432.15	1283.41	5.99